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CALTRAIN SAN FRANCISCO
DOWNTOWN EXTENSION PROJECT
CONCEPTUAL DESIGN AND DRAFT EIS/EIR

Geotechnical Engineering Recommendations Report

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December 27, 1995
Job No. 30751-001-003

ICF Kaiser Engineers, Inc.
1800 Harrison Street
Oakland, CA 94612-3430

Attention: Mr. David Minister
Senior Project Manager

Report
Geotechnical Engineering Recommendations
CalTrain S.F. Downtown Station Relocation
San Francisco, California

We are pleased to submit 40 copies of the referenced report presenting the results of our engineering analyses and recommendations.

The work was undertaken at the request of ICF Kaiser Engineers, Inc. and is in accordance with our proposal dated April 10, 1995 and our Contract No. 65928-SC-03 with ICF Kaiser Engineers, Inc., and with Amendments No. 1 and No. 2 thereof.

A preliminary draft of this report was issued on September 6, 1995 in preparation for the second meeting with the Board of Special Consultants. The major engineering issues and recommendations have been reviewed and agreed on by the Board members.

A draft of this report, dated October 6, 1995, was issued for review by representatives of the Joint Powers Board, and by ICF Kaiser Engineering and other team members. In addition, Professor Thomas O'Rourke, Chairman of the Board of Special Consultants, has reviewed the draft report and provided us with his comments.

The comments provided by the various reviewers have been considered carefully and the report modified as required to address the review comments.

If you have any questions with regard to the contents of this report, we would be pleased to discuss them with you.

Very truly yours,

DAMES & MOORE

D. Koutsoftas
Principal

TABLE OF CONTENTS

Page

1.0 INTRODUCTION 1

2.0 PROPOSED CONSTRUCTION 3

3.0 PURPOSE AND SCOPE 6

4.0 INTERPRETED SUBSURFACE CONDITIONS AND GEOTECHNICAL PARAMETERS FOR DESIGN 7
  4.1 SUBSURFACE STRATIGRAPHY 7
  4.2 GEOTECHNICAL PARAMETERS FOR DESIGN 9
    4.2.1 Geotechnical Characterization of the Fill Stratum 10
    4.2.2 Characterization of the Bay Mud 10
    4.2.3 Characterization of Upper Sands 13
    4.2.4 Characterization of Old Bay Clay 13
    4.2.5 Characterization of Lower Sands with Clay Lenses 14
    4.2.6 Characterization of Older Alluvial and Bay Deposits 14
    4.2.7 Bedrock 14
    4.2.8 Groundwater Conditions 15

5.0 FOUNDATIONS OF EXISTING STRUCTURES 16

6.0 EXCAVATIONS AND SHORING 19
  6.1 EXCAVATION FOR TERMINALS 19
    6.1.1 Excavations for the Beale/Market Station 20
    6.1.2 Excavation for the Beale/Mission Station; Alternatives 2B and 2C 23
    6.1.3 Excavations for Alternative 3: Terminal at Transbay Transit Terminal . . 25
  6.2 EXCAVATIONS ALONG THE RAIL LINE ALIGNMENTS 26
    6.2.1 Alignments 2A, 2B, 2C, Cut-and-Cover 26
      6.2.1.1 Excavations Along Beale Street 26
      6.2.1.2 Excavations Along the Embarcadero 28
      6.2.1.3 Excavations Along Townsend (King) 29
    6.2.2 Excavation Along Alternatives 3A/3B 30
      6.2.2.1 Excavations North of Folsom Street 30
      6.2.2.2 Excavations Along Colin P. Kelly Jr. Street 30
      6.2.2.3 Extension of Cut-and-Cover Alignment Between Fifth and Seventh Streets 31
  6.3 SPECIAL CONDITIONS 31
    6.3.1 Excavations Along the Embarcadero Roadway 32
    6.3.2 Excavation Under the Harrison Street Bridge 35
    6.3.3 Excavations at West Bay Bridge Anchorage 36
      6.3.3.1 Seventy-five-foot-wide Excavations 36
      6.3.3.2 Forty-foot-wide Excavations 38

7.0 TUNNELING 40
7.1 ALTERNATIVE TUNNELING ALIGNMENTS 40
7.2 TUNNELING METHODS AND CONSTRUCTION ISSUES 41
7.2.1 Tunneling in Soft Ground 41
7.2.2 Dewatering 42
7.2.3 Other Construction Issues 43
7.2.4 Excavation Staging 43
7.3 DESIGN CONSIDERATIONS 44
7.4 CONCLUDING REMARKS 46

8.0 FOUNDATION SUPPORT 48

9.0 IMPACTS OF EXCAVATIONS AND TUNNELING 49
9.1 GROUND MOVEMENTS CAUSED BY EXCAVATIONS 49
9.1.1 Excavations Involving Soft Bay Mud 49
9.2 GROUND MOVEMENTS CAUSED BY TUNNELING 51

LIST OF REFERENCES AND BIBLIOGRAPHY 54

APPENDIX - BOARD OF CONSULTANTS REPORTS

LIST OF PLATES

Plate No.	Title
1-1	Study Corridor and Vicinity Map
2-1	Plan of Alternative Alignments
4-1	Map of Original Shoreline and Marshes with Proposed Alternative Alignments
4-2	Subsurface Profile - Yerba Buena Cove Area: Along Beale Street
4-3	Subsurface Profile - South Beach Area: Along the Embarcadero Roadway
4-4	Subsurface Profile - Mission Bay Area: Along Townsend Street
4-5	Subsurface Profile - Yerba Buena Cove Area: Through Transbay Terminal Site
4-6	Results of Field Vane Shear Tests Performed in Yerba Buena Cove Along Beale Street
4-7	Results of Field Vane Shear Tests Performed in Mission Bay Along Townsend Street
4-8	Results of Field Vane Shear Tests Performed in Mission Bay Near Seventh and Berry Streets
4-9	Excavatability of Rock
6-1	Excavation and Bracing System, Beale Street Station Excavations: Market to Howard Streets
6-2	Comparison of Shoring Systems: BART and CalTrain Stations
6-3	Evaluation of Base Stability - 35 Foot Deep Excavation: Beale/Market Street Station
6-4	Evaluation of Base Stability - Final Excavation Depth: Market/Beale Street Station. Alternative 2A-C&C

6-5 Relative Positions of Adjacent Building Foundations to the Beale/Market Street Excavations

6-6 Construction of a Jet-Grouted Slab to Control Deflections of the Shoring System and Settlements. Alternative 2A-C&C

6-7 Potential for Undermining Existing Pile Foundations During Slurry Trench Excavation

6-8 Excavation and Bracing System for the Beale/Market Street Station: Alternative 2B

6-9 Excavation Shoring System and Base Stability for the Beale/Mission Street Station: Alternatives 2B and 2C

6-10 Staged Excavation to Improve Excavation Stability: Alternatives 2B and 2C

6-11 Excavation and Shoring System, CalTrain Terminal, Alternative 2C, Excavations Near Howard Street

6-12 Excavation and Shoring System; Transbay Terminal: Alternative 3B

6-13 Excavation and Shoring System; Transbay Terminal: Alternative 3B

6-14 Excavation and Bracing System for Excavations Between Howard and Folsom Streets: Alternative 2A-C&C

6-15 Excavation and Bracing System; Alternatives 2B-C&C and 2C-C&C: Excavations Near Folsom Street

6-16 Excavation and Shoring System; Underground Walkway Between Mission and Market Streets

6-17 Excavation and Shoring System in the Vicinity of Harrison Street: Alternative 2A-C&C

6-18 Excavation and Shoring System - South Beach Area: Alternative 2A-C&C

6

6

6-

6-

6-1

R:W

6-19	Excavation and Shoring System Along Townsend Street: Areas Between First and Colin P. Kelly Jr. Streets and Between Third and Fourth Streets: Alternative 2T
6-20	Excavation and Shoring System in Area in the Vicinity of Second and Third Streets Along Townsend: Alternative 2T
6-21	Excavation Shoring System: Alternative 3B, Near Howard Street
6-22	Excavation Shoring System: Alternatives 3A/3B, Near Brannan Street
6-23	Excavation Shoring System: Alternatives 3A/3B-T Between Fifth and Sixth Streets
6-24	Excavation Shoring System: Alternatives 3A/3B-T At Sixth Street
6-25	Excavation Shoring System: Alternatives 3A/3B-T Near Berry Street
6-26	Cross Section Showing Excavation and Adjacent Structures Along the Embarcadero
6-27	Soft Ground Tunneling Alternative Along the Embarcadero
6-28a	Staged Excavation Alternative: Cut-and-Cover Tunneling Along the Embarcadero (Steps 1 & 2)
6-28b	Staged Excavation Alternative: Cut-and-Cover Tunneling Along the Embarcadero (Steps 3 & 4)
6-29	Excavation Under Harrison Street Bridge: Alternative 2A-C&C; Deep Excavations
6-30	Excavation under Harrison Bridge: Alternative 2B-C&C; Shallow Excavations
6-31	Cut-and-Cover Excavation at West Anchorage of Bay Bridge: Alternatives 2A-C&C

6-32a Stage I Excavation, Alternative 2A-C&C Option A. Cut-and-Cover Construction: Bay Bridge Anchorage Area

6-32b Stage II Excavation, Alternative 2A-C&C Option A. Cut-and-Cover Construction: Bay Bridge Anchorage Area

6-32c Stage III Excavation, Alternative 2A-C&C Option A. Cut-and-Cover Construction: Bay Bridge Anchorage Area

6-33a Stage I Excavation, Alternative 2A-C&C Option B. Cut-and-Cover Construction: Bay Bridge Anchorage Area

6-33b Stage II Excavation, Alternative 2A-C&C Option B. Cut-and-Cover Construction: Bay Bridge Anchorage Area

6-34 Excavation at Bridge Anchorage; Alternative 2A-C&C Option C

6-35 Excavation at Bay Bridge Anchorage; Two Track Alternative (Narrower and Shallower Excavations)

7-1 Plan of Tunneling Alternatives

7-2 Tunnel Profile: Alternative 2A-MT

7-3a Tunnel Profile: Alternative 3B-T (Folsom to Brannan)

7-3b Tunnel Profile: Alternative 3A-T (Folsom to Brannan)

7-4 Tunnel Profile: Alternative 3AB-T (Long Tunnel: Bryant to Folsom)

7-5 Partial Tunnel Profile: Alternative 3A-T (Tunnel under Historic Buildings)

7-6 Partial Face Excavation Double Track Tunnel

7-7 Rock Stabilization By Grouting Below Bench

7-8 Pressure Diagrams for the Design of Tunnel Supports and Permanent Liner

9-1 Settlements and Lateral Deformations Caused by Excavations

9-2 Settlements Caused by Tunneling

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DRAFT REPORT
GEOTECHNICAL ENGINEERING RECOMMENDATIONS
CALTRAIN S.F. DOWNTOWN STATION RELOCATION

1.0 INTRODUCTION

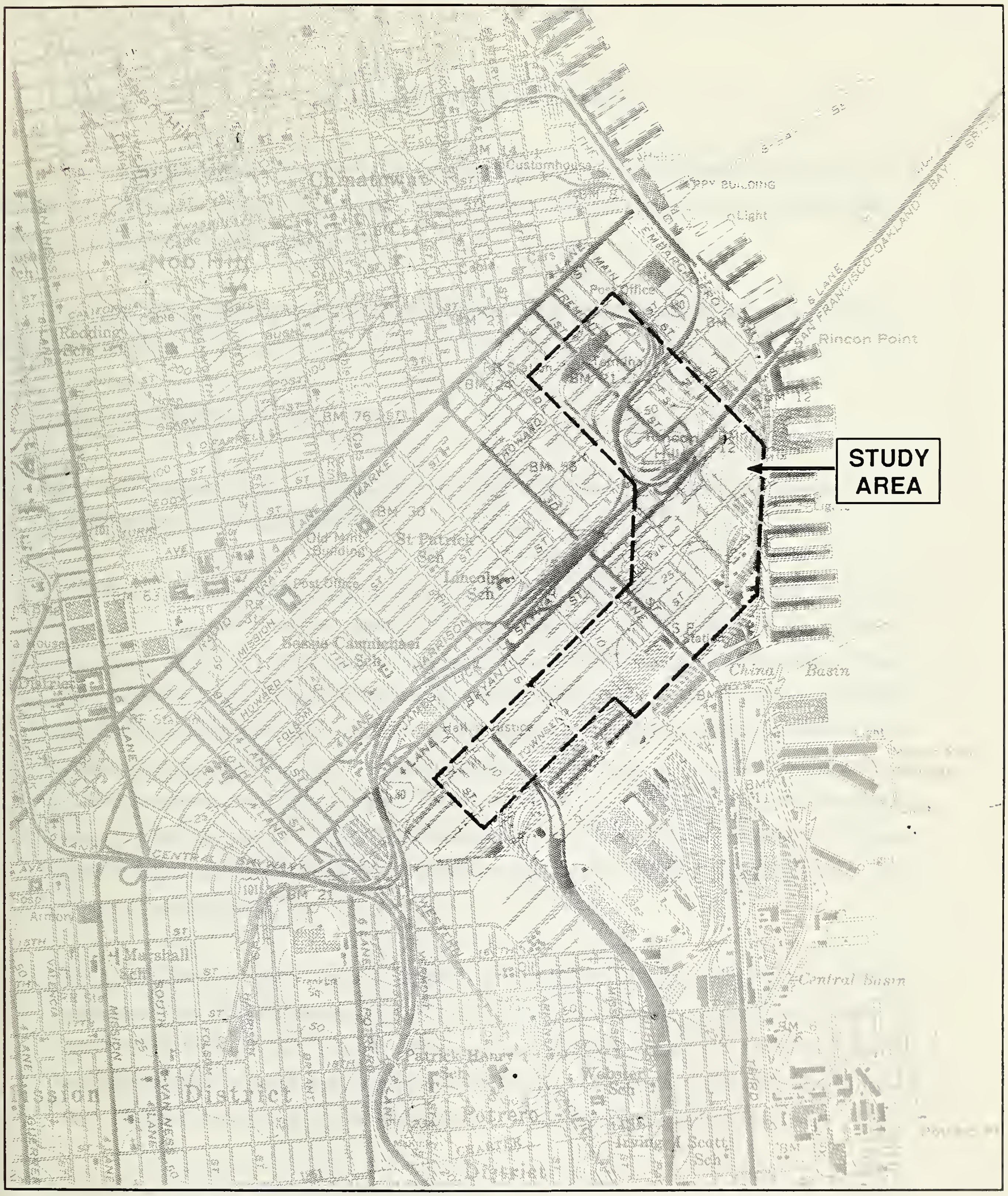
This report presents the results of the Geotechnical Studies carried out between April and October 1995 for the proposed CalTrain S.F. Downtown Station Relocation project. The geotechnical studies are part of the EIS/EIR effort for the CalTrain project. The work was performed at the request of ICF Kaiser Engineers, Inc. and is in accordance with the scope of services outlined in our proposals dated April 10, 1995 and July 25, 1995, and ICF Kaiser Engineers, Inc. Contract No. 65928-SC-03 and Amendments 1 and 2.

The site of the study corridor is located south of Market Street, as shown on Plate 1-1. In the north-south direction, it extends from Market to King Street, between Beale and Second Streets. In the east-west direction, it covers a strip extending from the waterfront along the Embarcadero to Sixth Street, between King and Brannan Streets. Between Sixth and Seventh Streets, the routes merge into a single route that connects with the existing CalTrain route along Seventh Street.

The study involves evaluation of various alignments in terms of the environmental impacts and the technical and cost feasibility of various construction alternatives, including cut-and-cover tunneling under existing streets, as well as mined or bored tunnels along portions of the various alignments. In addition to the construction of the railroad line required for the proposed relocation, the project involves construction of a downtown station. Presently, three options are under consideration for the station. The first involves a deep underground 1,000-foot-long station along Beale Street between Howard and Market Streets that would tie into the BART Embarcadero station. The second involves a shallower underground station along Beale Street between Mission and Folsom Streets; and the third option is to construct a station at the site of the existing Transbay Transit Terminal.

The results of the site investigation performed for this project have been summarized in the "Geotechnical Site Investigation Report" dated September 25, 1995, which provides detailed information on the subsurface conditions along each of the various alignments considered. This report addresses issues relating to the design and construction of the proposed railroad line extension and the underground stations.

Following this Introduction, Section 1, the report is organized as follows: Section 2 provides a brief description of the proposed construction; Section 3 defines the purpose and scope of the investigation; Section 4 reviews the interpreted subsurface conditions and the geotechnical characterization of the major strata; Section 5 reviews the foundation conditions of major structures that could be affected by the proposed construction; Section 6 presents recommendations for excavation and shoring design; Section 7 addresses technical issues related to tunnels and associated construction methods; Section 8 discusses foundation support requirements; and Section 9 reviews the impacts of excavations and tunneling on existing structures and other facilities.



Source: U.S.G.S. 7.5 minute San Francisco North quadrangle, 1973.

STUDY CORRIDOR AND VICINITY MAP

December 1995
30751-001-003

ICF Kaiser Engineers, Inc.
Caltrain S.F. Downtown Station
Relocation EIS/EIR Project

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PLATE 1-1

2.0 PROPOSED CONSTRUCTION

The Peninsula Corridor Joint Powers Board (JPB) plans to extend the Peninsula Commute Service (PCS) from its present remote terminal at Fourth and Townsend Streets to the Financial District in downtown San Francisco.

At present the following alternative alignments and station locations are being considered, as shown on Plate 2-1.

1. CalTrain Terminal Station Locations

1.1 Market/Beale Street: an approximately 1,000-foot-long, 3-level cut-and-cover subway under Beale Street (alternative 2A).

1.2 The CalTrain Downtown San Francisco Terminal would be located all on *one* level below ground in a staggered track and platform arrangement between Market and Folsom Streets, using both the Beale Street right-of-way and the vacant Caltrans right-of-way east of Beale Street. Two of the CalTrain terminal tracks would extend underground all the way to Market Street with a mezzanine above. The other two CalTrain terminal tracks would be located at the same elevation as the first two, but would only extend as far as Mission Street, using the Caltrans right-of-way east of Beale Street. An attractive underground mezzanine would connect passenger movement from the two sets of tracks and would also connect the Mission Street tracks to the BART/MUNI Embarcadero Station at Market and Beale Streets. This is part of alternative 2B.

1.3 The CalTrain Terminal would be located all on one level below ground between Mission and Folsom Streets, using both the Beale Street right-of-way and the vacant Caltrans right-of-way east of Beale Street. An attractive underground mezzanine with a moving sidewalk would connect the north end of the CalTrain terminal at Mission and Beale Streets to the BART/MUNI Embarcadero Station at Market and Beale Streets (alternative 2C).

1.4 The CalTrain Downtown Station would be located on the second level of a new Transbay Transit Terminal (TTT) to be constructed at the site of the existing terminal, after the existing TTT has been demolished (alternative 3A).

1.5 The CalTrain Terminal would be located one level below ground in a new Transbay Transit Terminal, which would be constructed at the site of the existing terminal after the existing TTT has been demolished (alternative 3B).

2. CalTrain North-South Alignments

2.1 Along Beale Street from Market Street to the Embarcadero to be constructed by cut-and-cover tunneling techniques. This is part of alternatives 2A-C&C, 2B-C&C and 2C-C&C. For ease of reference, the cut-and-cover portion of the alignment south of the station will be referred to as alternative 2A-C&C.

2.2 Along Beale Street from Market to Harrison Streets constructed by cut-and-cover tunneling techniques. Southward from Harrison Street, a mined or bored tunnel in a sweeping, approximately 3,000-foot radius curve under private property to either Third and Townsend Streets or Third and King Streets. This is part of alternative 2A-M.T.

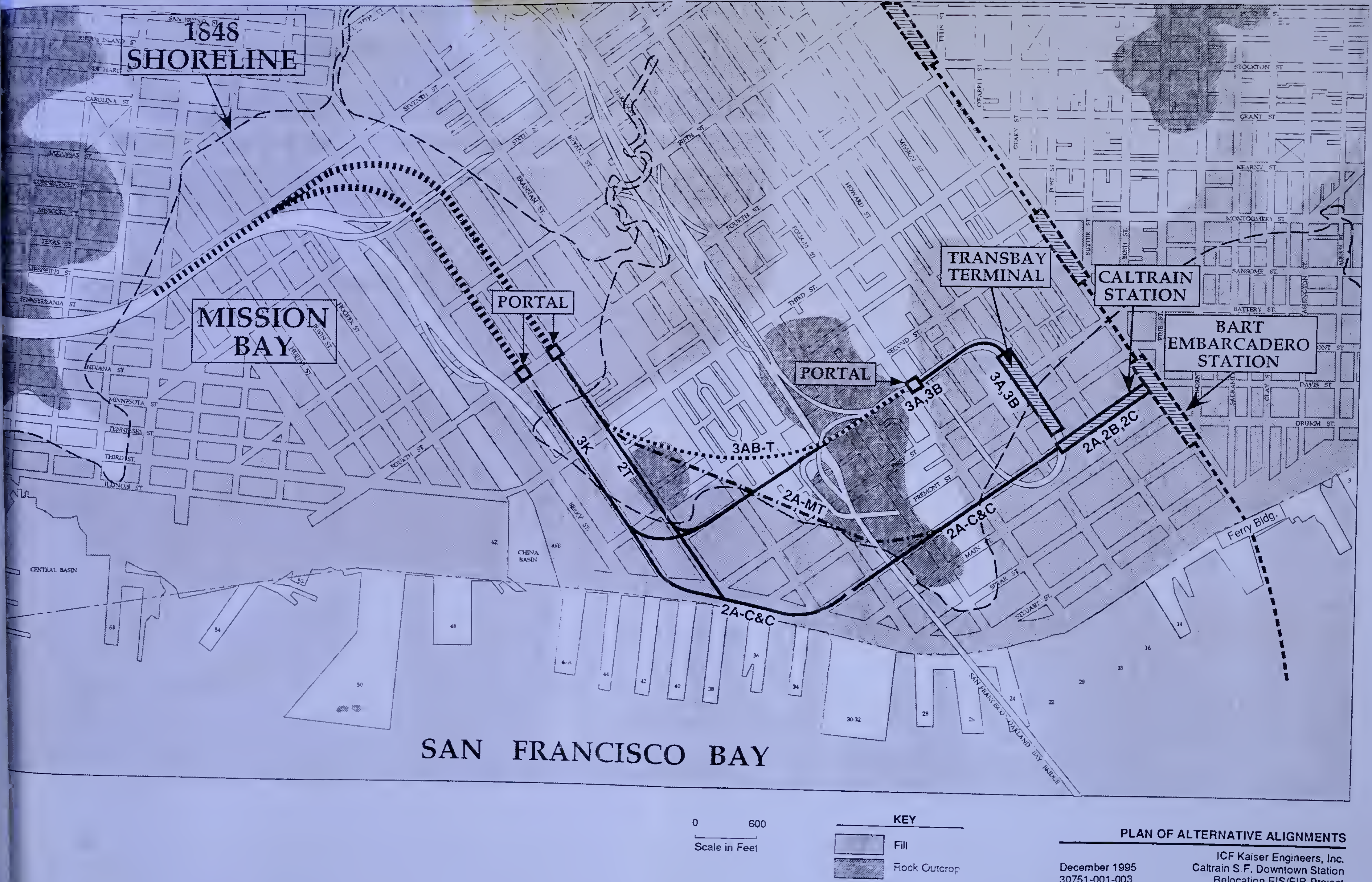
2.3 Colin P. Kelly Jr./Essex Street Alignment from Howard Street to either King or Townsend Street. The portion of this alignment between Folsom and Townsend would be constructed as a bored or mined tunnel under private property and the remainder as a cut-and-cover. The length of the tunnel portion remains to be determined. The portal would be near Folsom Street. The tunnel portion would connect to an aerial structure that would extend north to the existing Transbay Transit Terminal. This is part of alternative 3A. Another variation of this alternative involves an underground line north of Folsom Street connecting to an underground station at the Transbay Transit Terminal, one level below ground (alternative 3B). Two options are being considered for the portion of the alignment that will be constructed as a mined tunnel. The first alternative would involve tunneling between Folsom Street to the north and a point midway between Brannan and Bryant Streets at the south end. The second option would involve a longer tunnel that extends from Folsom Street to Bryant Street under Essex and Colin P. Kelly Jr. Streets, and then follows a sweeping curve under private property between the intersection of Colin B. Kelly Jr. and Bryant Streets and the intersection of Third and Townsend Streets (stations 172+30 and 195+00). This is referred to as alternative 3A&3B-T.

3. CalTrain East-West Alignments

3.1 Along Townsend Street (the Locally Preferred Alignment Previously Selected) from Seventh Street to the Embarcadero, and then along the Embarcadero to the north-south

alignment along Beale Street. This portion would be constructed by cut-and-cover tunneling methods with the portal between Fifth and Fourth Streets and is referred to as alternative 2T. Another alternative would also connect to the Colin P. Kelly Jr. alignment, referred to as 3A-T or 3B-T. An extended subway with a portal near Seventh and Berry Streets will also be considered (longer subway through Mission Bay).

3.2 Along King Street from Seventh Street to the Embarcadero, and then along the Embarcadero to the north-south alignment along Beale Street. This portion would be constructed by cut-and-cover tunneling methods, with a portal between Fourth and Fifth Streets, and is referred to as alternative 3K. An extended subway with a portal near Seventh and Berry Streets will also be considered (longest subway segment through Mission Bay).



PLAN OF ALTERNATIVE ALIGNMENTS

December 1995
30751-001-003

ICF Kaiser Engineers, Inc.
Caltrain S.F. Downtown Station
Relocation EIS/EIR Project

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PLATE 2-1

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3.0 PURPOSE AND SCOPE

The purpose of our study is to evaluate the subsurface conditions along the various alignments, develop parameters for design, and prepare recommendations for conceptual design purposes so that the feasibility, constructibility and costs associated with each of the various alternatives can be evaluated as part of the EIS/EIR effort.

To achieve the objectives of the program, we undertook the following scope of services:

- Reviewed and evaluated in detail the subsurface conditions along each alignment, and divided each alignment into various segments of similar subsurface conditions or where similar construction methods could be employed to construct the tunnels and excavations.

- Performed preliminary engineering assessments for excavations and tunnels and developed recommendations for excavation and shoring.

- Assessed alternative tunneling methods and their applicability to the ground conditions encountered.

- Reviewed and evaluated the broad issues associated with the impacts of the proposed construction on existing structures and facilities.

- Coordinated, prepared and attended two meetings with the Board of Special Consultants, whose responsibility is to advise the team on issues of technical feasibility, constructibility and costs for the various alternatives being considered.

- Prepared this report summarizing our findings and recommendations.

4.0 INTERPRETED SUBSURFACE CONDITIONS AND GEOTECHNICAL PARAMETERS FOR DESIGN

The subsurface conditions along the various alignments were reviewed in a companion report entitled "Geotechnical Site Investigation, CalTrain S.F. Downtown Relocation," Vols. I and II dated September 25, 1995. For ease of reference, some of the more important aspects of the subsurface stratigraphy are reviewed below, with emphasis on those characteristics that influence the selection of construction methods and the performance of the shoring systems and the tunnels.

4.1 SUBSURFACE STRATIGRAPHY

The CalTrain alignments traverse areas of distinctly different geologic characteristics. As shown on Plate 4-1, the cut-and-cover alignment indicated as alternative 2A-C&C (also 2B-C&C) and 2T (for the portion along Townsend Street) traverses the old Yerba Buena Cove, Rincon Hill, the Embarcadero waterfront along the former South Beach area, the former high ground area near Steamboat Point, and the former Mission Bay

The subsurface conditions in the Yerba Buena Cove area, South Beach, and Mission Bay are dominated by the presence of soft and compressible marine soils, known locally as San Francisco Bay Mud. The depth of the sediments over bedrock in these former bays and coastline varies substantially depending on the position of the area of interest relative to the old shoreline. Yerba Buena Cove, Mission Bay and South Beach have been reclaimed from the Bay by filling in the late 1800s and early 1900s. The fill materials used for the reclamation varied substantially from dune sand to rubble, building demolition debris, and other materials. Along portions of these areas abandoned ships and old wharves have been buried in the fill.

The portions of the corridor that are inland from the old shoreline, such as portions of alternative 2 that go through Rincon Hill and Steamboat Point, will encounter rock at the surface or at shallow depths. Bedrock is of the Franciscan formation, a material consisting of graywacke sandstone, shales and other sedimentary rocks that have been altered by the processes of chemical and physical weathering as well as by tectonic movements along a series of active and inactive faults between the San Andreas fault on the west side and the Hayward fault on the east side of the Bay. The rock is generally of exceptionally poor quality because of the intense shearing and weathering.

Detailed subsurface sections have been prepared for each of the alignments under consideration. This information is presented in the "Site Investigation Report." Rather than attempt to repeat much of this information, this section discusses characteristic profiles for the areas in Yerba

Buena Cove, South Beach, and Mission Bay. These profiles are shown on Plates 4-2, 4-3, 4-4 and 4-5. The profile on Plate 4-5 is for the Transbay Transit Terminal and, as can be seen from Plate 2-1, the terminal extends from inland of the old shoreline to a significant distance outboard of the old shoreline.

As all of these profiles show, the subsurface conditions are as follows:

- The surface soils are loose to medium dense FILL, approximately 15–30 feet thick. The fill contains rubble and other debris, but a significant portion of the Fill consists of dune sand excavated from the dunes along Market Street and dumped indiscriminately to fill the Bay.

- Below the fill layer is the Recent BAY MUD, whose thickness is also quite variable. Along Beale Street, the greatest thickness, up to 40 feet, is found between Mission and Market Streets. South of Mission Street there is a gradual reduction in thickness until the Mud pinches out entirely near Folsom Street (where Beale Street intersects the old shoreline). Along the Embarcadero waterfront the Bay Mud is typically less than 20 feet thick, while in the Mission Bay area the thickness varies from essentially zero near Fourth Street to more than 70 feet between Fifth and Sixth Streets along Townsend Street. The thickness of the Mud decreases toward Seventh Street to 30 feet or less.

- The Bay Mud is underlain by a layer of brown and orange very dense SAND, known locally as the COLMA formation (see Schlocker, 1974). The thickness of this layer varies considerably. It is found consistently along Beale Street and along the Embarcadero roadway. It is also found along portions of the east-west alignments along Townsend and King Streets, east of Fourth Street. In the Mission Bay area, sand lenses are found between the Bay Mud and the underlying older Bay deposits but the sand lenses appear to be relatively loose marine sediments and do not belong to the Colma formation.

- Below the Colma and/or the loose Marine Sands is a layer of stiff to very stiff fat clay referred to locally as the OLD BAY CLAY. The thickness of this stratum varies considerably and is more predominant in areas occupied previously by the Bay. Along Beale Street (in the former Yerba Buena Cove area) the Old Bay Clay is up to 125 feet thick. However, its thickness decreases in a southerly direction, and pinches out just north of Folsom Street. In the South Beach area, along the Embarcadero and

along portions of the alignment along Townsend Street, the Old Bay Clay is interbedded with sand layers and its thickness is quite variable. Because of the difficulty in differentiating the Old Bay Clay consistently from the sand layers over this portion of the alignment, these deposits are collectively referred to as Older Bay or Older Alluvial Deposits. They represent fairly stable, competent strata. The frequency of the sand lenses increases with depth. In the Mission Bay Area the Old Bay Clay varies in thickness from being completely absent near the fringes of the Bay to being more than 50 feet thick.

- Along portions of the alignment, the Old Bay Clay and Older Bay and Alluvial Deposits are underlain by a Colluvial/Alluvial layer over Bedrock.

4.2 GEOTECHNICAL PARAMETERS FOR DESIGN

As far as the performance of excavations and underground box structures is concerned, the Recent Bay Mud is the most important stratum to consider in the geotechnical design and construction. The strength and deformation characteristics of the Bay Mud are critical in terms of the base stability of the excavations, ground deformations caused by the excavations, and the response of adjacent structures to the deformations caused by the excavation.

The fill layer is also important in terms of base stability of excavations, ground deformations, and dewatering. The firm deposits below the Bay Mud are of lesser significance, particularly where the excavations are 45 feet deep or less. However, where deeper excavations, up to 83 feet, are being considered, as for the Beale/Market Street station (alternatives 2A, 2B and 2C), the strength characteristics of the Old Bay Clay would be important in terms of the base stability of the excavation and ground deformations.

With regard to tunneling, since the tunnel portion of the alignment is planned to be constructed in rock, the Franciscan rock formation is obviously the key stratum. Rock cover above the tunnels, rock quality in terms of strength, discontinuities and weathering of the rock mass, and in-situ permeability are important factors in evaluating issues of constructibility. They are also critical for evaluating the influence of tunneling on the stability and deformation of structures that are directly above or adjacent to the tunnel alignment.

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4.2.1 Geotechnical Characterization of the Fill Stratum

The fill thickness along the various alignments currently being considered is quite variable. In the Rincon Hill area, along Essex Street, the fill is typically less than 3 feet and consists of gravelly road base material. The fill is also relatively thin along Townsend and King Streets in the area of the former Steamboat Point, where the rock is found at shallow depths.

The fill is significantly thicker in the areas of the former Yerba Buena Cove, South Beach and Mission Bay. The fill thickness typically increases outboard of the old shoreline. Fill thicknesses range from 10 to 30 feet.

The fill is characterized as loose to medium dense, fine to medium sand with variable amounts of silt. It contains obstructions including rubble, concrete, bricks and other building debris, trash and wood, including wood piles from old abandoned wharves. For engineering analyses and design, we recommend the following parameters:

Parameter	**Value**
Total Unit Weight	120 pcf
Effective Friction Angle	32°
Hydraulic Conductivity	1×10^{-3} to 5×10^{-2} cm/sec
Coefficient of Earth Pressure at Rest	0.47
Design Groundwater Level	10 feet

The fill is quite susceptible to liquefaction. Where fill is encountered below the groundwater table, moderate to strong shaking should be expected to cause liquefaction and associated settlements and lateral spreading.

4.2.2 Characterization of the Bay Mud

The Bay Mud is a compressible fat clay (classified as CH according to the Unified Classification System), ranging from medium stiff near the top of the layer to stiff near the base of the deposit. It contains various amounts of shells, sand and organic matter, particularly near the top of the layer. In the Mission Bay Area, distinct layers of sandy Bay Mud were found at depths of 20 to 30 feet below the top of this stratum.

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Undrained Strength Characteristics

The Bay Mud is known to follow normalized behavior with regard to its undrained shear strength characteristics. Furthermore, the Bay Mud is quite anisotropic, which means that the undrained shear strength (Ladd and Foott 1974, Ladd et al. 1977) varies with the direction of loading. The strength is highest where the soil is sheared by increasing the vertical stress (active condition), and has the least strength when it is sheared by increasing the horizontal stress (passive condition). When it is sheared along a horizontal plane, it has intermediate strength. For normally consolidated Bay Mud, the following normalized strength ratios are appropriate.

Compression: $S_u/\bar{\sigma}_v = 0.30$

Direct Simple Shear: $S_u/\bar{\sigma}_v = 0.23$

Extension: $S_u/\bar{\sigma}_v = 0.18$

Undrained strengths measured with the field vane are essentially the same as strengths measured with the direct simple shear test.

The effects of overconsolidation can be accounted for by the following equation.

$$S_u = S \times \bar{\sigma}_{vo} \times (\mathrm{OCR})^{0.85}$$

where S_u is the undrained shear strength.

S	is the appropriate $S_u/\bar{\sigma}_v$ value for normally consolidated soil
$\bar{\sigma}_v$	is the vertical effective stress
$\bar{\sigma}_{vo}$	is the in-situ effective overburden stress
OCR	is the overconsolidation ratio = $\bar{\sigma}_{vmax}/\bar{\sigma}_{vo}$; and
$\bar{\sigma}_{vmax}$	is the maximum past pressure from consolidation tests.

The undrained shear strength of the Bay Mud increases with depth. Undrained strengths measured at five field vane shear test locations are summarized on Plates 4-6 to 4-8. The vane shear strength is a reliable index test. Values of vane shear strengths need to be corrected to account for anisotropic and rate effects using empirical correction factors (Bjerrum, 1972; Mesri, 1989; Ladd et al., 1977). For the Bay Mud a correction factor of 0.9 is typically used. The corrected vane shear strengths are very close to the strengths determined from SHANSEP principles and $S_u/\bar{\sigma}_v$ values

determined from direct simple shear tests Along the portions of the alignment that traverse the Old Yerba Buena Cove area, the Bay Mud is typically stiff, with strengths in the range of 850 to 1,000 psf. For preliminary conceptual design purposes, a shear strength value of 900 psf is recommended.

The results of the vane shear strength tests performed at two locations along Beale Street (in the area of the Yerba Buena Cove) indicate that the Mud is preconsolidated. The preconsolidation is likely to have been induced by construction dewatering and perhaps by pumping from the sand layers below the Mud to withdraw water for commercial purposes in the early 1900s. Notable construction activities that might have induced consolidation of the Mud due to dewatering include construction of the BART system and various high-rise buildings with deep basements.

We have no relevant information for the South Beach area. However, based on our experience, we would anticipate that the mud in this area is normally consolidated, because of the proximity of this area to the Bay and due to the absence of notable construction activities that could have impacted the Bay Mud. The strength of the Bay Mud can be estimated from normalized strength behavior. For preliminary design purposes we recommend strength values estimated from:

$$S_u = 400 + 9\,(d - 20)$$

where d is the depth below ground level; all strength values expressed in pounds per square foot.

In the Mission Bay Area, the Mud may be assumed for the most part to be normally consolidated. However, at the location of vane shear tests FV-4 and FV-5, the top 20 feet of the Mud appear to be lightly overconsolidated with measured vane strengths of about 750 psf. For design purposes, the strength of the mud in the Mission Bay area, at least along Townsend Street, may be assumed to be constant at the top 20 feet (depth interval at 20 feet to 40 feet) at about 650 psf, and increasing linearly with depth below a depth, d, of 40 feet as follows:

$$S_u = 675 \text{ psf} + 9(d - 40)$$

Other Parameters

For preliminary engineering purposes, we recommend the following parameters for the Bay Mud:

Parameter	Value
Total Unit Weight	100–105 pcf

Coefficient of Earth Pressure at Rest	0.55
Hydraulic Conductivity	1×10^{-7} cm/sec

4.2.3 Characterization of Upper Sands

The upper sands consist of a medium dense dark gray Clayey Sand, of marine deposition, referred to as the Marine Sand, and a rust brown and tan medium-to-fine Sand, known as the Colma Sand.

The Gray Sand is typically less than 10 feet thick and is essentially absent in the area south of Rincon Hill. The Gray Sand is quite silty and clayey and may contain stringers of clay. For engineering analysis, a total unit weight of 120 pcf is recommended and an effective friction angle of 32°, with no effective cohesion.

The Colma sand is typically dense to very dense with standard penetration N values in the range of 40 to 80 blows per foot. It is a medium-to-fine sand with variable amounts of silt, and is slightly cemented. The cementation is highly variable. For preliminary design purposes, we recommend a total unit weight of 130 pcf and an effective friction angle of 35°, with no effective cohesion.

4.2.4 Characterization of Old Bay Clay

Extensive deposits of Old Bay Clay are present in the area north of Rincon Hill, and in the South Beach area toward the waterfront as well as in the Mission Bay area. The maximum thickness of this layer occurs in the area between Mission and Howard Streets, where it is up to 125 feet thick. The Old Bay Clay is a low to medium plasticity clay classified as CL or CH according to the Unified Soil Classification System. The Old Bay Clay is interbedded with sand lenses of variable thicknesses and horizontal extent. Based on the available information, it is not possible to identify reliably the areas and depth of the sand lenses. A total unit weight of 115 pcf is recommended for this layer.

The Old Bay Clay is overconsolidated. Numerous consolidation tests performed for various projects in the vicinity of the study corridor, as well as tests performed for this study, indicate overconsolidation ratios in the range of 1.5 to 2.

The Old Bay Clay is stiff to very stiff in consistency with undrained shear strength that increases approximately linearly with depth below the top of this layer. For preliminary engineering

analysis, a strength of 1,500 psf can be assumed at a depth of 80 feet increasing at a rate of 20 psf per foot of additional depth.

4.2.5 Characterization of Lower Sands with Clay Lenses

The lower sands are highly variable and appear to consist of a sequence of sediments that vary from fairly clean medium-to-fine sands to clayey sands. Within the lower sands there are layers of very stiff clays, similar to the Old Bay Clay, although the deeper clay layers contain trace to little fine gravel, suggesting that these sediments may be of Alluvial origin, at least in the areas inboard of the old shoreline. The strength of these very stiff clays probably exceeds 2,500 psf. A total unit weight of 120 pcf may be used for the clay layers.

The sands are very dense, with standard penetration resistances ranging from 60 to over 100 blows per foot. A total unit weight of 130 pcf is recommended for calculation purposes. An effective friction angle of 35° and no effective cohesion is recommended for analysis.

4.2.6 Characterization of Older Alluvial and Bay Deposits

The older Alluvial and Bay deposits identified on the cross-sections along the east-west portions of the various alternatives are very similar to the lower sands with clay lenses. They consist of very dense sands interbedded with very stiff clays. The sands vary considerably in gradation, from fine relatively clean sands, to silty sands and clayey sands. The strength parameters and other soil parameters recommended for the lower sands with clay lenses may be used for these strata as well.

4.2.7 Bedrock

The bedrock consists of interbedded Shales, Sandstones, sheared Shale Melange, and occasional Siltstone. The depth to rock varies from as little as one to two feet to over 200 feet.

The rock varies considerably from very weak highly sheared and weathered shales, and very low to medium-strong sandstone. Unconfined compressive strengths of relatively *intact* specimens of shale range from 180 to 5,800 psi. Unconfined compression tests on *intact* specimens of sandstone indicate strengths ranging from as low as 1,000 psi up to 27,000 psi. The samples, which had very low strengths, failed along discontinuities in the rock specimens, which form planes of weakness.

The results of the point load index tests show considerable variation in the results from as low as 16 psi to 638 psi for Sandstone; and from 6 psi to 213 psi for Shale. The results of the point load

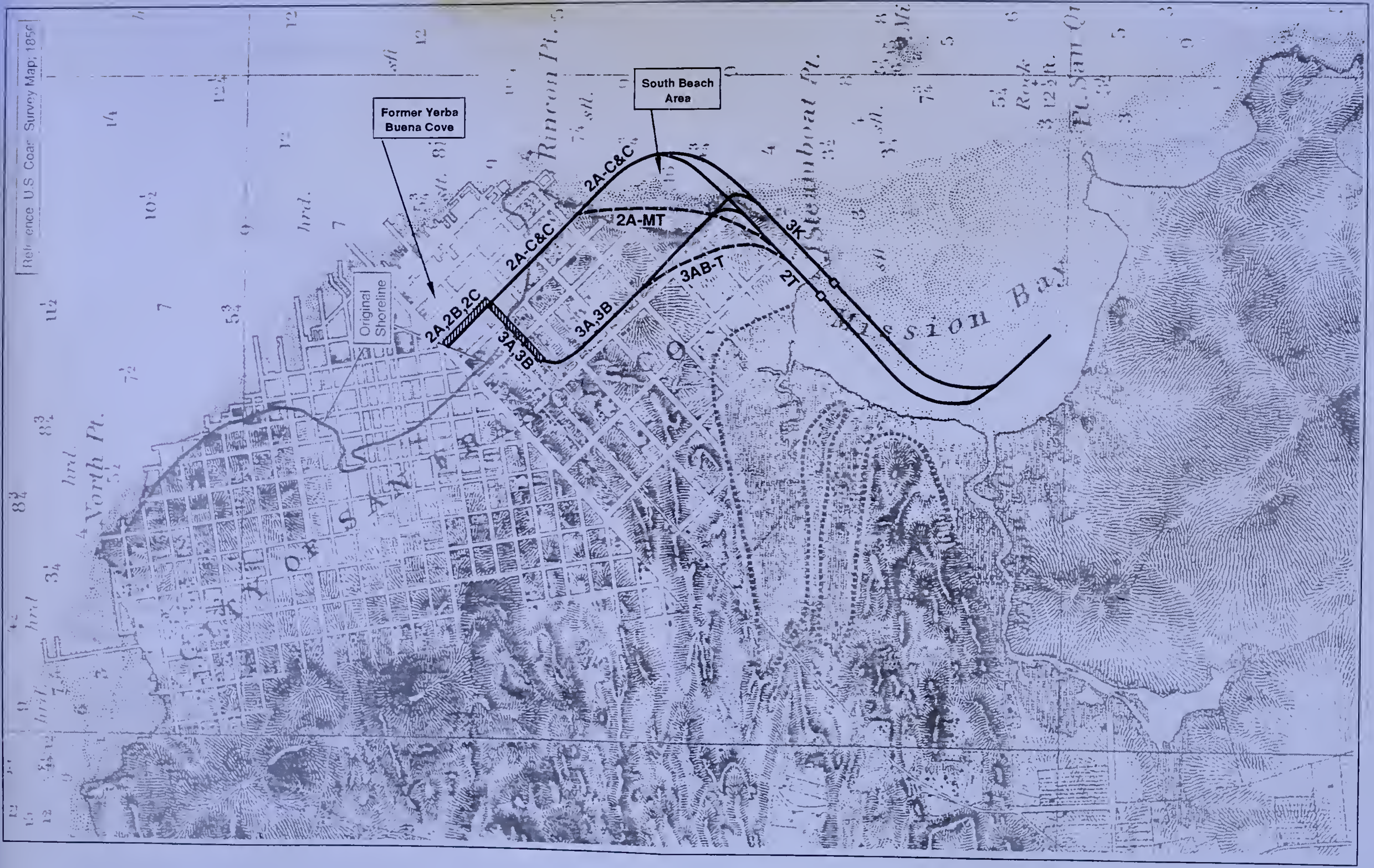
index tests were correlated with fracture spacing, within each segment of the core from which the test samples were selected. Plate 4-9 shows the results of the tests plotted on a chart that includes guidelines for excavatability of rock (Naval Facilities Engineering Command Manual, 1982). The results of the tests indicate that the majority of the rock can be excavated either by digging or by ripping. However, it should be expected that zones of fairly massive rock would be encountered that may require blasting to loosen the rock.

It must be emphasized that the majority of the rock recovered from the boreholes was highly sheared with considerable degree of weathering. However, there are zones within the weathered, sheared and fractured rock that are fairly intact. Based on the information from the 8 boreholes drilled into rock for this study, it is estimated (for the preliminary purposes of this study) that the zones of intact rock represent less than 20% of the rock mass. It should be noted, however, that significant additional coring would be required to estimate reliably the percentage of sound rock that may be encountered along each of the alignments.

4.2.8 Groundwater Conditions

The groundwater table elevations measured in the boreholes drilled for this investigation varied from elevation -3 to -10 feet (CCSF datum) in the low-lying areas. In the high areas, on Rincon Hill, groundwater levels vary substantially with measurements in the range of +35 to +45 feet. It would appear that the groundwater levels in the higher elevations represent perched water tables above the Shale and Siltstone layers that may be the result of leakage from underground utilities. Further investigation, however, will be needed to verify this assessment.

In the area along the Embarcadero roadway, because of the proximity to the Bay, the groundwater levels should be on the average about 10 feet below ground level and should fluctuate with the tides.



MAP OF ORIGINAL SHORELINE AND MARSHES
WITH PROPOSED ALTERNATIVE ALIGNMENTS

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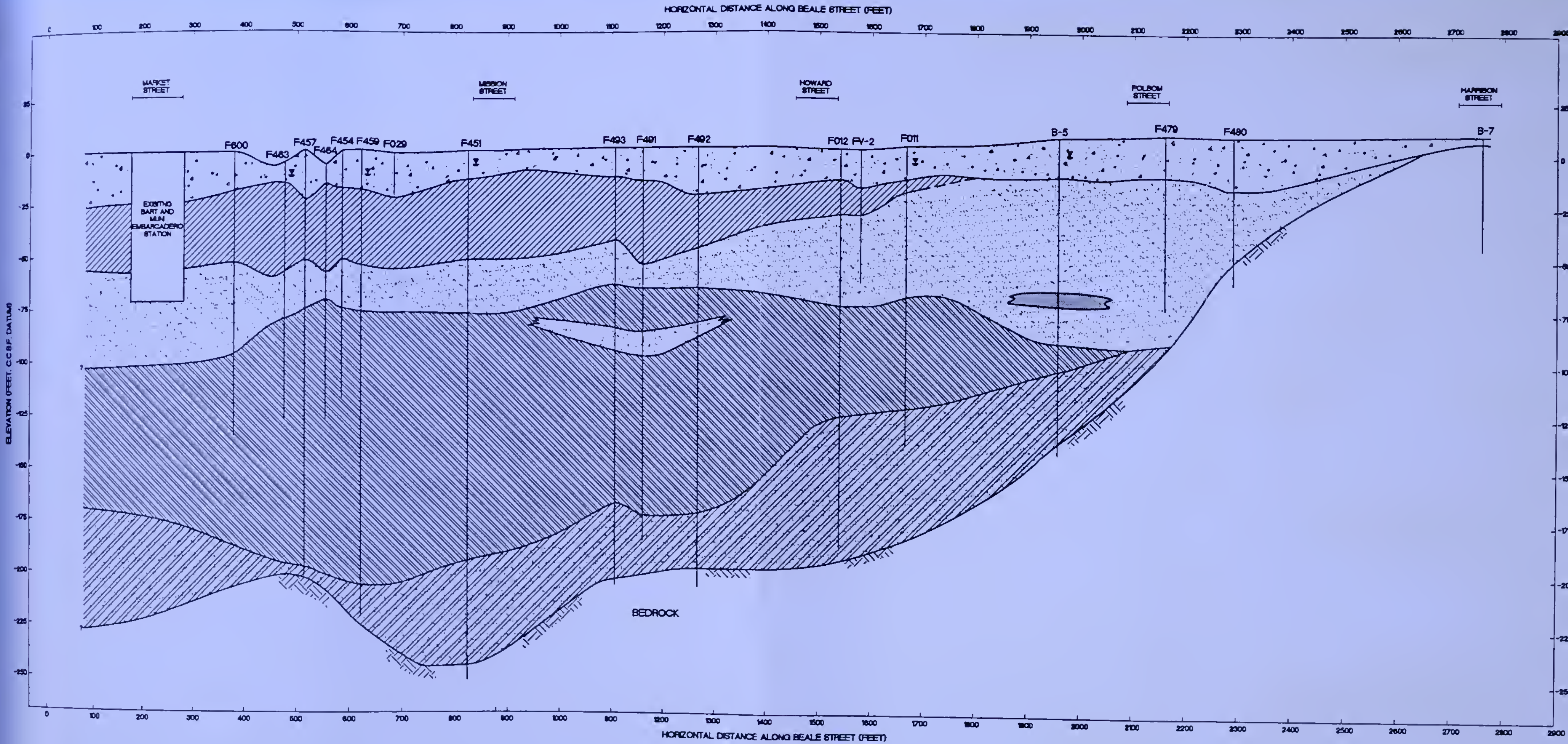
PLATE 4-1



LEGEND

- ⁘ FILL
- ▨ BAY MUD
- ⁘ UPPER SANDS WITH CLAY LENSES
- ▧ OLD BAY CLAY
- ▨ LOWER SANDS WITH CLAY LENSES
- ▧ MEDIUM STIFF SILTY CLAY
- ▩ STIFF CLAY

NOTES

1. THE SUBSURFACE SECTION SHOWN REPRESENTS OUR EVALUATION OF THE MOST PROBABLE CONDITIONS BASED ON INTERPRETATION OF AVAILABLE DATA, LOCAL GEOLOGY AND OUR GENERAL EXPERIENCE IN THE AREA. THE SECTIONS ARE BASED ON INTERPOLATION AND EXTRAPOLATION OF DATA AND INVOLVE INCREASING UNCERTAINTY WITH DISTANCE FROM BORING LOCATIONS SHOWN ON PROFILE. IN PREPARING THIS CROSS-SECION BORINGS HAVE BEEN PROJECTED PERPENDICUALR TO THE SECTION.
2. FOR DETAILED SOIL INFORMATION SEE BORING LOGS IN APPENDIX A.
3. FOR DETAILED DESCRIPTIONS OF THE MAJOR STRATA SEE THE MAIN TEXT OF THE REPORT.
4. GROUND SURFACE ELEVATIONS ARE BASED ON TOPOGRAPHIC MAPS PROVIDED BY AGS INC.

SUBSURFACE PROFILE
YERBA BUENA COVE
AREA: ALONG BEALE STREET

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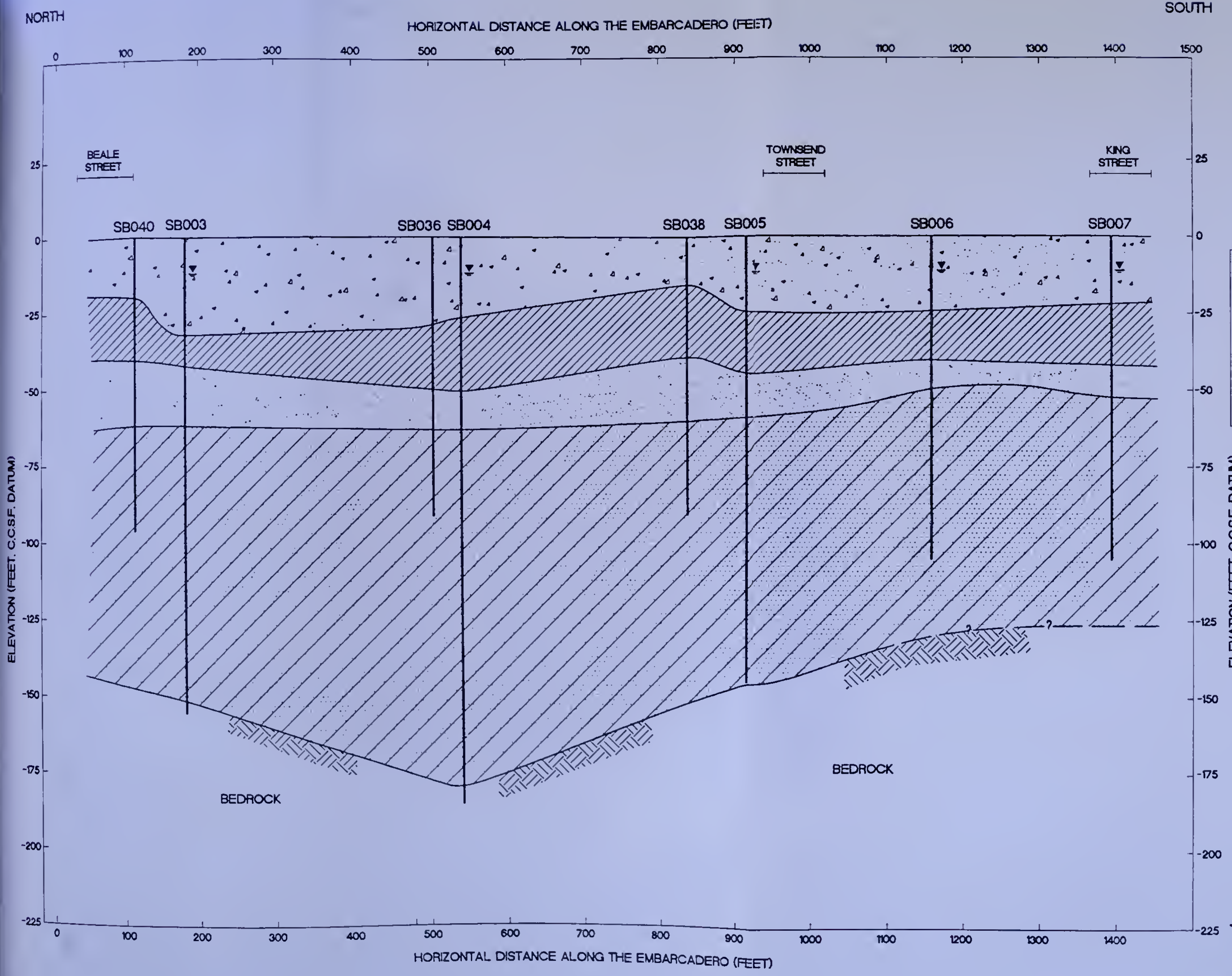
PLATE 4-2



LEGEND

- △ FILL
- ▨ BAY MUD
- ⁘ UPPER SANDS WITH CLAY LENSES
- ▧ OLDER BAY DEPOSITS: INTERBEDDED DENSE SANDS AND STIFF CLAYS

NOTES:

1. THE SUBSURFACE SECTION SHOWN REPRESENTS OUR EVALUATION OF THE MOST PROBABLE CONDITIONS BASED ON INTERPRETATION OF AVAILABLE DATA, LOCAL GEOLOGY AND OUR GENERAL EXPERIENCE IN THE AREA. THE SECTIONS ARE BASED ON INTERPOLATION AND EXTRAPOLATION OF DATA AND INVOLVE INCREASING UNCERTAINTY WITH DISTANCE FROM BORING LOCATIONS SHOWN ON PROFILE. IN PREPARING THIS CROSS-SECION BORINGS HAVE BEEN PROJECTED PERPENDICUALR TO THE SECTION.
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4. GROUND SURFACE ELEVATIONS ARE BASED ON TOPOGRAPHIC MAPS PROVIDED BY AGS INC.

SUBSURFACE PROFILE
SOUTH BEACH AREA:
ALONG THE EMBARCADERO ROADWAY

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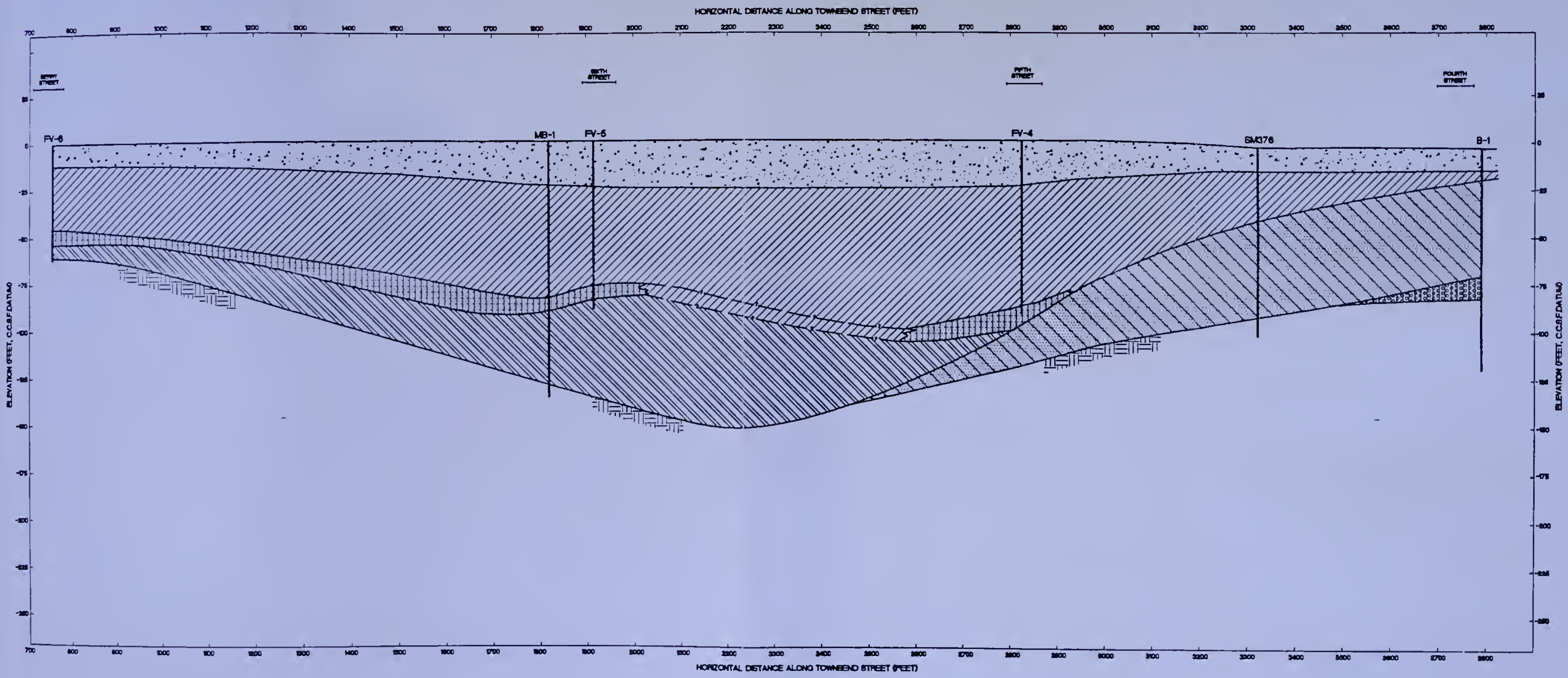
PLATE 4-3



LEGEND

⁘	FILL
▨	BAY MUD
▥	LOOSE TO MEDIUM DENSE SILTY TO CLAYEY SAND (MARINE SAND)
▧	OLD BAY CLAY
⧄	OLDER BAY DEPOSITS: INTERBEDDED DENSE SANDS AND STIFF CLAYS
⧅	OLDER ALLUVIAL DEPOSITS: DENSE SANDS, SILTS, AND STIFF CLAYS
⚬	BROWN CLAYEY GRAVEL (COLLUVIUM)

NOTES:

1. THE SUBSURFACE SECTION SHOWN REPRESENTS OUR EVALUATION OF THE MOST PROBABLE CONDITIONS BASED ON INTERPRETATION OF AVAILABLE DATA, LOCAL GEOLOGY AND OUR GENERAL EXPERIENCE IN THE AREA. THE SECTIONS ARE BASED ON INTERPOLATION AND EXTRAPOLATION OF DATA AND INVOLVE INCREASING UNCERTAINTY WITH DISTANCE FROM BORING LOCATIONS SHOWN ON PROFILE. IN PREPARING THIS CROSS-SECTION BORINGS HAVE BEEN PROJECTED PERPENDICULAR TO THE SECTION.
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3. FOR DETAILED DESCRIPTIONS OF THE MAJOR STRATA SEE THE MAIN TEXT OF THE REPORT.
4. GROUND SURFACE ELEVATIONS ARE BASED ON TOPOGRAPHIC MAPS PROVIDED BY AGS INC.

SUBSURFACE PROFILE
MISSION BAY AREA:
ALONG TOWNSEND STREET

December 1995
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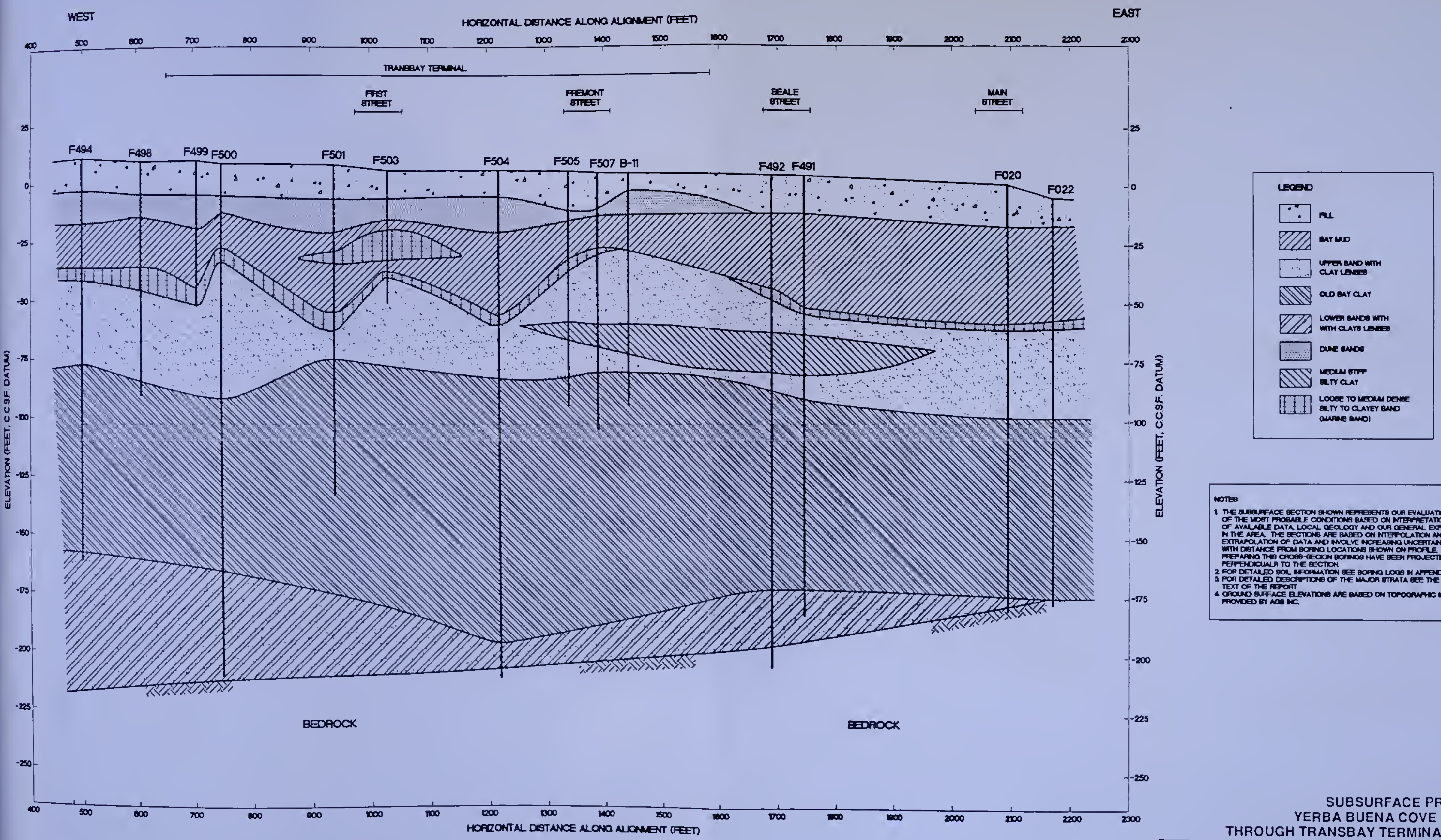
PLATE 4-4



NOTES:

1. THE SUBSURFACE SECTION SHOWN REPRESENTS OUR EVALUATION OF THE MOST PROBABLE CONDITIONS BASED ON INTERPRETATION OF AVAILABLE DATA, LOCAL GEOLOGY AND OUR GENERAL EXPERIENCE IN THE AREA. THE SECTIONS ARE BASED ON INTERPOLATION AND EXTRAPOLATION OF DATA AND INVOLVE INCREASING UNCERTAINTY WITH DISTANCE FROM BORING LOCATIONS SHOWN ON PROFILE. IN PREPARING THIS CROSS-SECION BORINGS HAVE BEEN PROJECTED PERPENDICUALR TO THE SECTION.
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4. GROUND SURFACE ELEVATIONS ARE BASED ON TOPOGRAPHIC MAPS PROVIDED BY AGS INC.

SUBSURFACE PROFILE
YERBA BUENA COVE AREA:
THROUGH TRANSBAY TERMINAL SITE

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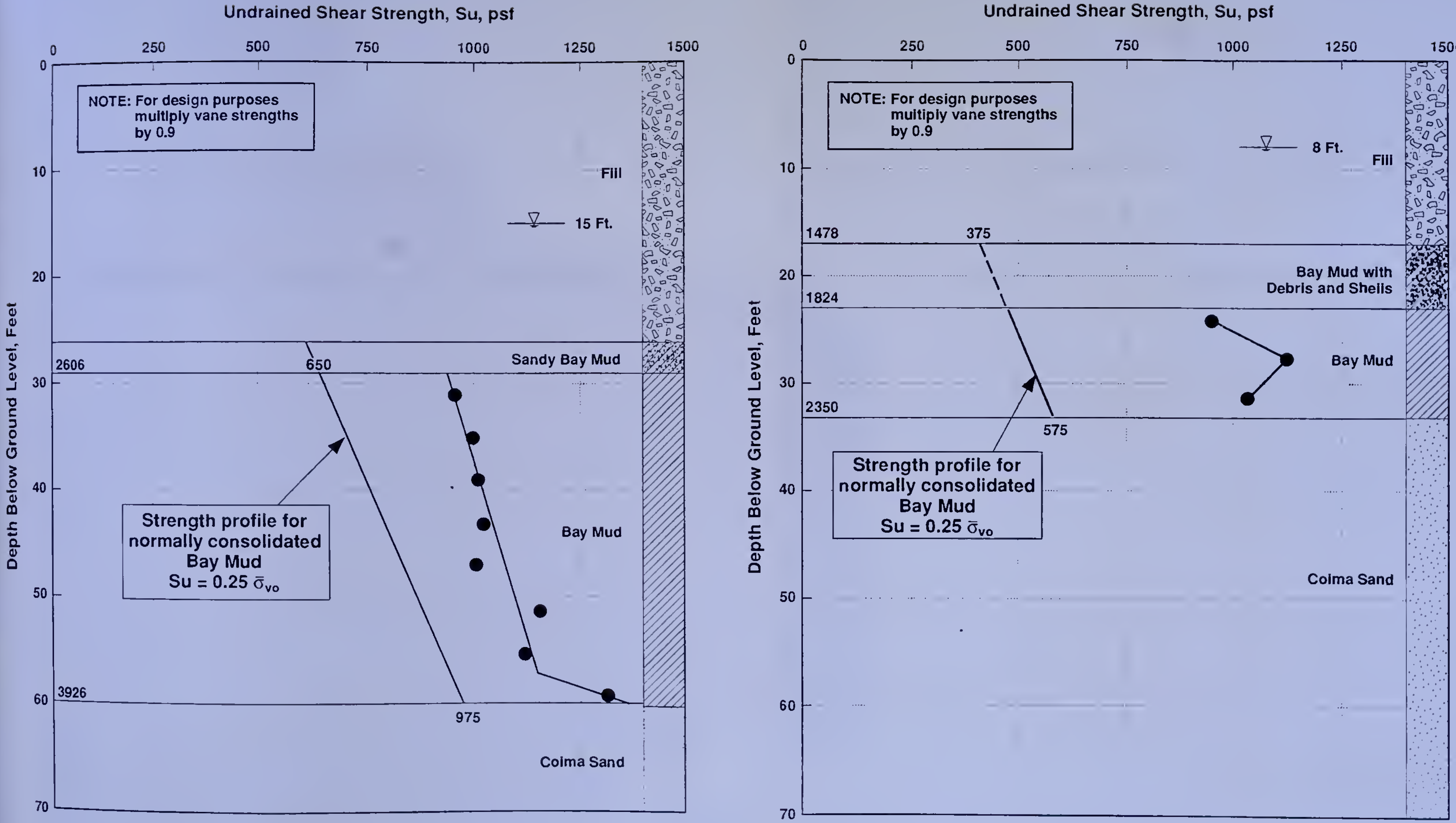
PLATE 4-5



(a) FIELD VANE: FV-3
1/2 BLOCK NORTH OF MISSION STREET

(b) FIELD VANE: FV-2
30 FT. SOUTH OF HOWARD STREET

RESULTS OF FIELD VANE SHEAR TESTS
PERFORMED IN YERBA BUENA COVE
ALONG BEALE STREET

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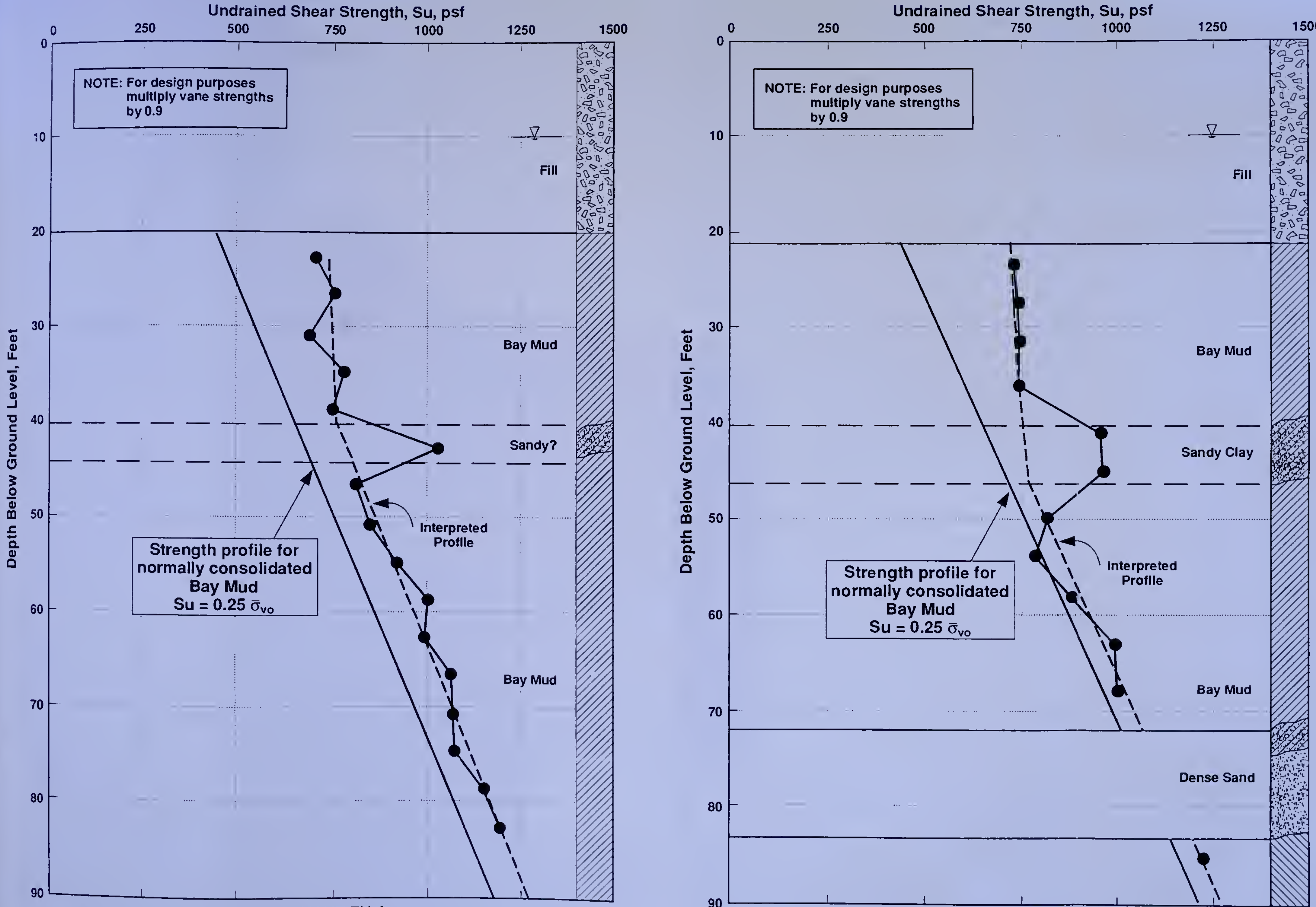

NOTE: For design purposes multiply vane strengths by 0.9

Strength profile for normally consolidated Bay Mud Su = 0.25 $\bar{\sigma}_{vo}$

a) FIELD VANE FV-4
5TH AND TOWNSEND STS.

b) FIELD VANE FV-5
6TH AND TOWNSEND STS.

RESULTS OF FIELD VANE SHEAR TESTS PERFORMED IN MISSION BAY ALONG TOWNSEND STREET

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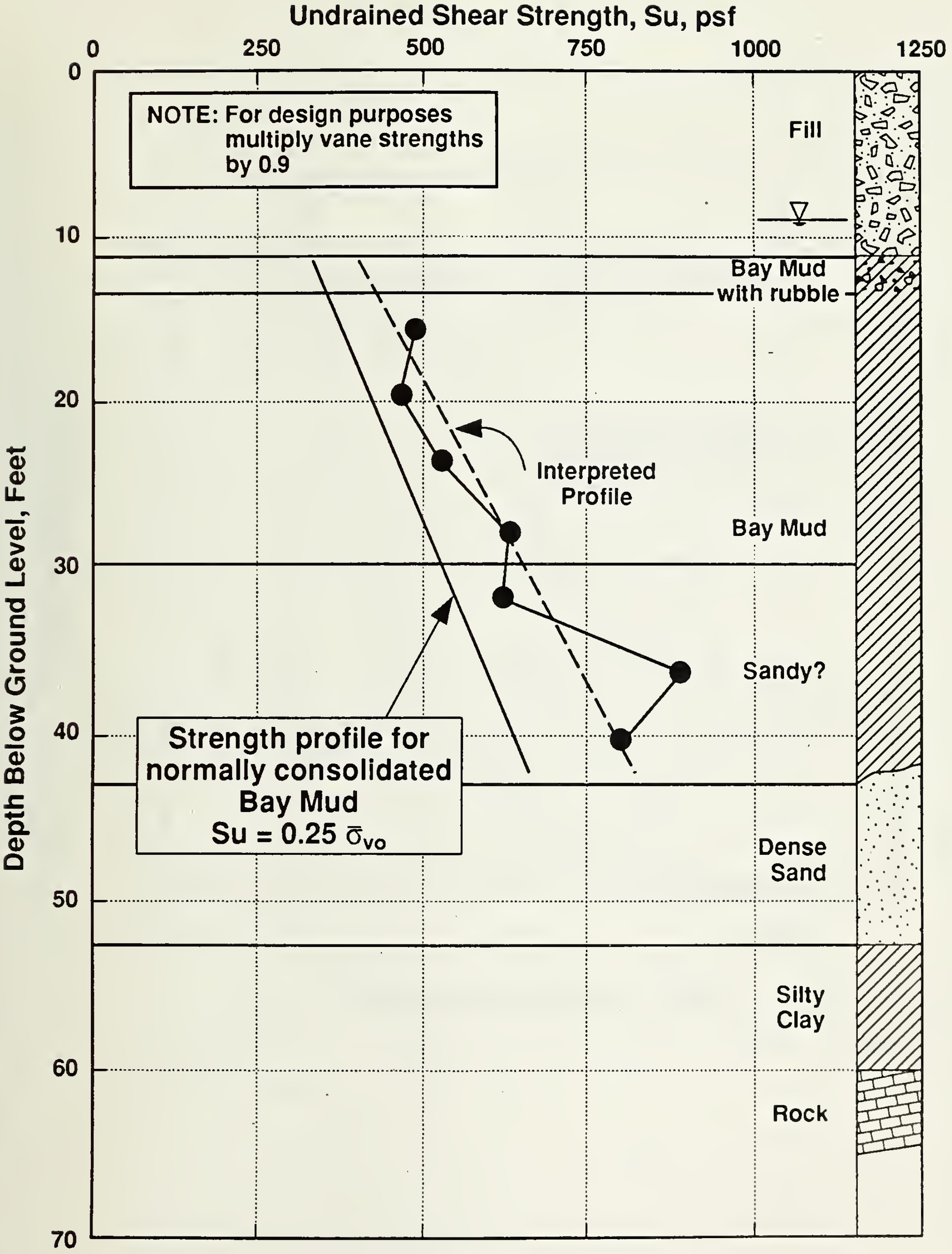
PLATE 4-7



FIELD VANE FV-6
ON BERRY ST. BETWEEN 7TH AND BERRY STS.

RESULTS OF FIELD VANE SHEAR TESTS PERFORMED IN MISSION BAY NEAR SEVENTH AND BERRY STREETS

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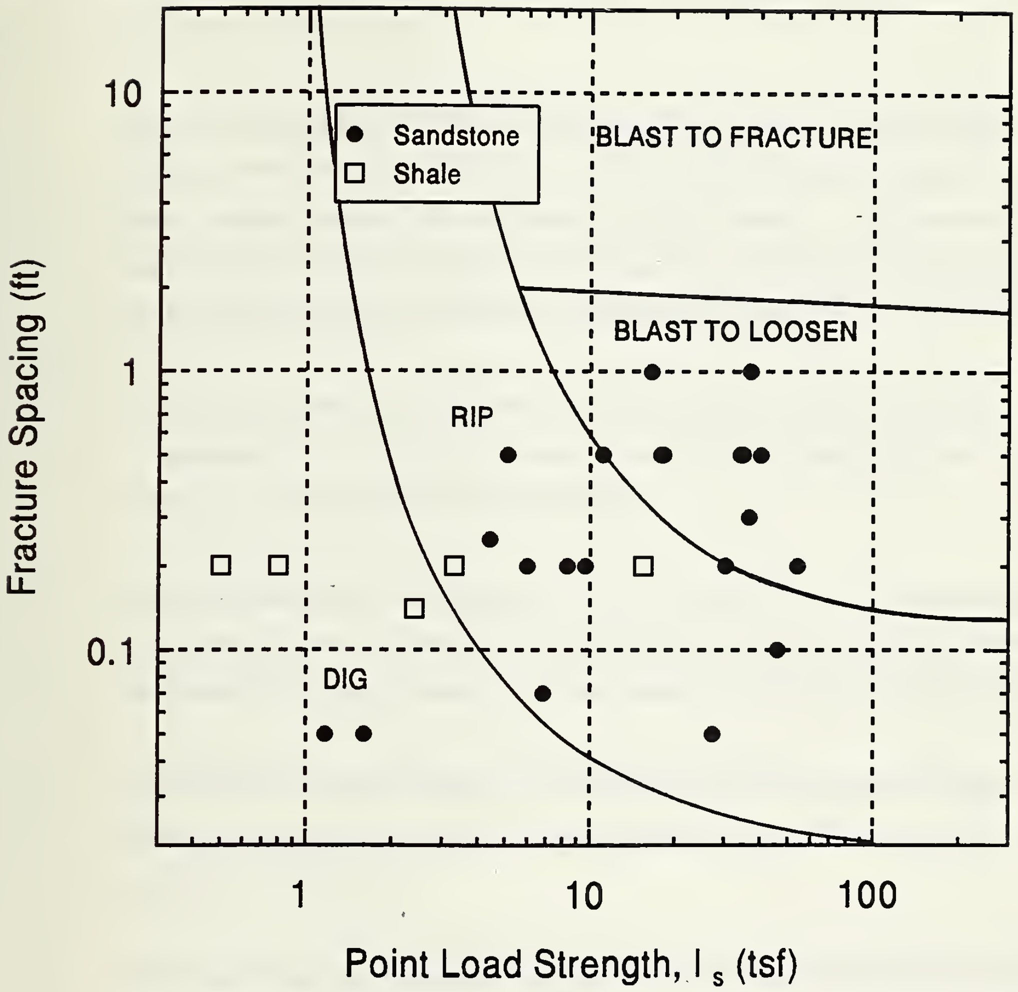

EXCAVATABILITY OF ROCK

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PLATE 4-9

5.0 FOUNDATIONS OF EXISTING STRUCTURES

Construction of the CalTrain S.F. Downtown Station Relocation Project will impact numerous structures, regardless of which alignment is eventually selected. The planned excavations will pass in many cases within a very short distance from very important structures. Included are several multistory structures including:

- The PG&E building (245 Market Street) at the southeast corner of Beale and Market Streets, constructed in 1923-1924. This building has been recently strengthened to meet modern criteria for earthquake-resistant design. It is supported on a large number of wood piles with their tips bearing in the upper sand strata at a depth of approximately 90 feet. The piles have been designed for a 27-ton load.

- The PG&E tower at the northeast corner of Beale and Mission Streets. This building is supported on 18-inch prestressed concrete piles penetrating into the upper sands with their tips at a depth of about 90 feet.

- The 333 Market Street building, a 32-story tower at the southwest corner of Market and Beale Streets. It is supported on 12-inch prestressed concrete piles with their tips embedded in the upper sands at about elevation -62 feet (CCSF datum).

- The 50 Beale Street building, a 23-story tower at the northwest corner of Beale and Mission Streets supported on 12-inch prestressed concrete piles with tip elevations at about -70 feet or so.

- The Pacific Gateway building along Mission Street between Beale and Main Streets. This building is supported on 12-inch-square prestressed concrete piles with approximate tip elevations of -72 feet.

- The 301 Howard Street building, at the southwest corner of Howard and Beale Streets. It is a 22-story tower supported on 12-inch-square prestressed concrete piles founded in the upper sands at about elevation -62 feet.

Other major structures that will be impacted by the proposed excavations include:

- The Harrison Street Bridge, over Beale Street, supported on spread footings founded on rock.

- The West Anchorage of the Bay Bridge. This is a massive structure founded in rock. The deepest portion of the foundation is at -27 feet (CCSF datum).

- The foundations of the on and off ramps of the I-280 Freeway at Townsend and Sixth Streets.

- A number of low-rise residential structures along the Embarcadero between Bryant and Townsend Streets These structures have been recently constructed and are supported on prestressed concrete piles penetrating into the Upper Sand strata.

- The 301 Mission Street building, at the corner of Mission and Beale Streets. It is a six-story, masonry building Although we have no relevant foundation data about this building, it is probably supported on wood piles penetrating through the Bay Mud into the Upper Sands.

- A six-story building on Beale Street just south of the 301 Mission building. It is likely to be supported on piles driven into the Upper Sands.

- Other important structures for which we have no reliable foundation data include the Fremont Center building, a six-story building just south and west of the 301 Howard building, and a four-story structure at the corner of Beale and Folsom Streets. It is very likely that these structures are supported on piles penetrating into the Upper Sands.

- The U.S. Post Office building that extends between Beale and Main Streets, just north of Harrison Street; and the Bay Crest Condominiums, a high-rise structure just south of Harrison Street. Both of these buildings are probably supported on shallow foundations founded on rock.

- The various tunneling alternatives being considered would pass under a number of two- to five-story, old, masonry buildings in the area of the study corridor between Bryant and Townsend Streets.

The excavations for the various alternatives range in depth from 42 to 83 feet. Some of the proposed alternatives will require excavations that would extend close to, and in some cases even below, the tips of the piles supporting these structures. Consequently, the design of the excavations

must consider carefully the impacts of the excavations and associated ground movements on these structures. Provisions must be made in the design to preserve their integrity.

In some areas, where the tunnels would pass under existing structures, the soil and rock cover above the tunnel would be relatively shallow. Again, the impact of the proposed tunneling activities must be considered carefully, and appropriate measures should be implemented to protect those structures that may be affected by tunneling activities.

A number of buildings along the cut-and-cover alternative along Beale Street (2A-C&C, 2B-C&C and 2C-C&C) have basements that extend under the sidewalks. The presence of the extended basements within the right-of-way would make construction more difficult, especially along the portion of the alignment between Mission and Market Streets. In some instances, the SPTC walls for the CalTrain excavations would abut against the basement walls. The design and construction methods must consider the effects of the CalTrain excavations on the extended basements.

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6.0 EXCAVATIONS AND SHORING

Each of the proposed alternatives involves a considerable length of the corresponding alignments that will have to be constructed using cut-and-cover tunneling techniques. Of particular importance are the excavations required to construct the terminals because of the significant depth and width of the cuts involved. However, the majority of the excavations required to construct the line are less than 45 feet deep and about 42 to 45 feet wide. The purpose of this section is to provide recommendations for the shoring systems that will be required. Our recommendations are based on consideration of excavation stability and consideration of potential impacts on adjacent structures and other facilities. The intent of these recommendations is to provide sufficient information for preliminary cost estimates. Detailed design criteria for shoring are not considered appropriate at this stage of the project. Therefore, the recommendations presented in this section of the report should be considered as our best evaluation based on limited analyses, our own experience and input from the Board of Special Consultants.

The remainder of this section presents recommendations for the terminals, followed by recommendations for the excavations for the line. Finally, special cases are examined and recommendations provided to deal with the special conditions that these cases represent.

6.1 EXCAVATION FOR TERMINALS

The present investigation is focused on three basic alternatives regarding the construction of the terminals in the financial district. The first option involves a station approximately 1,000 feet long to be constructed within the Beale Street right-of-way connecting to the BART and MUNI at the Embarcadero Station. The required excavation depth for this option is 83 feet.

The second option involves a station that terminates at Mission Street. It would be constructed with the tracks and platforms on a single level and a mezzanine above. It would be approximately 150 feet wide with approximately one-half of the station under the Beale Street right-of-way and the rest extending into the Caltrans right-of-way area just east of Beale Street. A variation of this alternative involves extension of a 2-track station under Beale Street to tie into the BART/MUNI Embarcadero Station.

The third option involves construction of the terminal at the location of the existing Transbay Transit Terminal. The existing structure would be demolished and a new building constructed to function as a rail and bus terminal. One of the alternatives being considered is to have the terminal

on the first floor, while the second alternative would involve construction of the rail terminal below ground, with the bus terminal and other associated facilities above ground.

6.1.1 Excavations for the Beale/Market Station

Plate 6-1 shows the subsurface section and recommended shoring system for the portion of the terminal between Mission and Market Streets where the Soft Bay Mud deposit is thickest. It involves construction of 3-foot-thick soldier pile and tremie concrete (SPTC) walls to a depth of 120 feet with the soldier piles extending deeper to a depth of 150 feet. The soldier piles would consist of W36×300 sections to provide the necessary stiffness to control ground deformations to relatively small values. The walls would be supported with heavy struts spaced 10 feet apart in the vertical direction. In the longitudinal direction the struts would be spaced 15 feet to 20 feet apart. The struts would have to be installed, without delay, as soon as the excavation had progressed 2 to 3 feet below each strut level, and each strut would be preloaded to predetermined levels to provide the required stiffness in the system to minimize deformations of the system.

The proposed shoring system is very similar to the shoring system used to construct the BART Embarcadero Station in the vicinity of Beale Street. Plate 6-2 compares the BART shoring system (on the left-hand side) to the proposed shoring system for the CalTrain Station. In both cases the SPTC walls extend approximately 40 feet below the base of the excavation. However, because the dense sand stratum is not as extensive in the area south of Market where the CalTrain Station is to be built, we propose that the soldier piles be extended below the SPTC wall by as much as 30 feet to provide additional support to resist the lateral pressures.

The stability of the base of the excavation, as well as the ground deformations that would be caused as the shoring system deflects in response to the removal of the soil between the walls, is of concern. The stability of the excavation is examined at two stages: (1) when the depth of the excavation reaches a critical level, at which heaving of the Soft Bay Mud is the controlling factor; and (2) when the excavation reaches full depth, when heaving of the Old Bay Clay layer is the controlling factor.

Plate 6-3 shows the geometry of the excavation and shoring system when the excavation reaches a depth of 35 feet. At this stage significant overburden would have been removed and the thickness of Bay Mud below the base of the excavation would be significant.

Potential failure surfaces and a schematic of the deflected slope of the walls are shown on Plate 6-3. A simplified but conservative analysis was adopted in this work for evaluating the stability

of the excavation base. It ignores the structural stiffness and strength of the wall, but recognizes the significance of the adhesion and friction between the embedded portion of the wall and the soil above the assumed failure surface. The factor of safety against basal heave failure is calculated from the following equation:

$$F.S. = \frac{Su \times Nc + 2(Su_d D/B)}{\gamma H + q}$$

where:

Su is the shear strength of the Bay Mud below the base of the excavation, taken as 900 psf;

Nc is a theoretical bearing capacity factor;

Su_d is the adhesion between the wall and the soil along the depth of the wall D, between the base of the excavation and the assumed failure surface;

B is the width of the excavation;

γ is the total unit weight of the various soils above excavation level;

H is the depth of the excavation; and

q is the surcharge load at the surface taken as a nominal value of 200 psf.

The results of the calculations are shown on Plate 6-3, and indicate a minimum factor of safety of 1.3, which is quite adequate. Lateral ground deflections at this stage of the excavation, given the calculated factor of safety of 1.3, are expected to be on the order of 1 inch or less. At this stage, impacts on major adjacent buildings would be minimal, because the buildings are supported on piles that penetrate deeper below the base of the excavation.

Plate 6-4 shows the excavation profile and subsurface conditions at the full depth of excavation. Potential failure surfaces are shown as well as the calculations for base stability. A factor of safety of 1.44 against base heave failure is estimated, ignoring the effects of surcharge due to loads from the piles supporting adjacent buildings. An approximate estimate of ground deformations indicates values of lateral deflections on the order of 2 inches or so, which is about 0.2% of the depth of the excavation.

The level of deformations estimated is of concern because of the proximity of several major buildings to the proposed excavation. As can be seen from Plate 6-5, the base of the excavation, when it reaches full depth, will extend below the tips of the concrete piles supporting the 50 Beale Tower, and is only a few feet above the tip of the piles for the old PG&E Building. If the full

surcharge effects of the loads imposed by the pile foundations supporting the adjacent buildings are considered in the evaluation of base stability, the factor of safety reduces from 1.4 to about 1.1. This is a very low factor of safety and it is our view that measures must be planned to minimize the lateral movements of the excavation walls at the level of the pile tips. Plate 6-6 illustrates the use of a jet-grouted strut to be constructed at a depth between 70 and 95 feet to strengthen the base and increase resistance against potential lateral movements of the wall.

Because of concerns with the effects of the deep excavations on adjacent buildings, alternatives 2B and 2C are under consideration, which would involve a different station configuration to reduce the depth of the required excavations from 83 feet to 55 feet. Such a reduction in the excavation depth would reduce the risks associated with the stability and deformations of adjacent structures considerably.

The construction of the SPTC walls is quite risky, however, because the pile foundations of the adjacent buildings are very close to the slurry trenches that must be excavated to construct the SPTC walls. Plate 6-7 illustrates the relative position of the slurry trench to the existing foundations. The stability of the trenches is affected by details of the construction procedures, and the consistency of the slurry, which is used to stabilize the trenches. We are very concerned about the portion of the trench to be excavated in the Upper Sands, in which the pile foundations of the adjacent buildings are embedded. The risk of collapse of the slurry trench in any one panel, although small, cannot be eliminated. Such a collapse could have serious impacts on the foundations of the adjacent buildings. If alternative 2A involving 83-foot-deep excavations is selected, which would require construction of SPTC walls, we recommend that the Upper Sands stratum be treated before the slurry trenches are excavated. The treatment would consist of grouting (most likely by permeation grouting techniques) a zone 20 feet wide, just outside the trench, for the full depth of the Upper Sands before excavating the slurry trench. This type of treatment would protect the pile foundations from the effects of the slurry trench excavation.

The concerns with the stability of the trench and the potential impacts on adjacent structures can be alleviated by constructing the walls using in-situ mixed soil-cement walls. Because this procedure does not create a trench and replaces whatever material is removed with cement grout, this procedure has little risk of adverse impacts on adjacent foundations. However, currently available equipment cannot construct walls deeper than 100 feet. Therefore, the use of soil-cement walls for the 83-foot-deep excavation is not recommended; but it could be used for the alternatives that involve shallower excavations.

The disadvantage of the soil-cement walls is that they are not as stiff as the concrete SPTC walls. This disadvantage can be overcome by constructing a jet-grouted slab between the walls, after the walls have been constructed but before excavation begins to brace the walls against each other, so that inward movement of the walls below the base of the excavation is minimal. For preliminary cost estimation purposes, we recommend the use of jet grouting as an approach designed to safeguard the sensitive, tall buildings adjacent to the excavation, especially those with pile foundations that terminate in the Colma Sand. It should be recognized, however, that the opportunity exists for using the results of additional site exploration in forthcoming phases of the project to refine stability assessments in this area and select alternative measures for controlling basal movements that would involve less cost. At this stage we recommend that a jet-grouted slab 20 feet thick be constructed from -55 feet to -75 feet (20 feet thick) to brace the walls. The thickness of the jet-grout slab is determined by the requirement of maintaining the width (*B*) to depth (*D*) ratio less than 4. In that manner, the slab would behave as a deep beam rather than a slab, avoiding the unwanted effects of bending of the slab in the transverse direction.

The use of the jet-grouted slab allows the use of a wall that is not as deep as would be required if the jet-grout slab was not used and makes the use of soil-cement walls technically feasible. The cost savings from using shallower and less stiff walls would compensate for the cost of the jet-grouted slab. Of course, the end result is that the soil-cement wall/jet-grouted slab combination provides a shoring system that is considerably less risky and capable of controlling deflections, with impacts to adjacent buildings within acceptable limits. If a jet-grouted slab was used, the depth of the walls could be reduced by 15-20 feet, as shown on Plate 6-8.

Plate 6-8 shows the cross-section of the shoring system for the shallower excavations (55 feet). This excavation is much more stable, and should have smaller deformations than the (83 feet deep) excavations for alternative 2A.

The shallower excavations would also allow use of shoring systems other than the concrete SPTC walls. It is recommended that in-situ mixed soil-cement walls be used to construct the walls rather than SPTC walls. The soil-cement walls have significant advantages. They are considerably less costly than the SPTC walls and create less disturbance to the ground. They are less risky in terms of the potential for settlements and impacts on adjacent structures.

6.1.2 Excavation for the Beale/Mission Station; Alternatives 2B and 2C

Plate 6-9 shows the subsurface stratigraphy and the arrangement of the proposed excavation and shoring system. As can be seen from this plate, the excavation is 150 feet wide and 55 feet deep.

At this location, the bottom of the Bay Mud stratum is a few feet above the base of the excavation. The Pacific Gateway Building foundations are within a few feet from the east excavation wall. The tips of the piles supporting the Pacific Gateway Building are about 17 feet below the base of the excavation but well within the area that could be affected by a potential failure surface, as shown in Plate 6-9. The bearing stresses imposed by the pile foundations have an adverse effect on the stability of the base of the excavation even though the SPTC shoring walls would extend 45 feet below the base of the excavation. It is our evaluation that extending the SPTC walls deeper is not an effective way to improve base stability. As can be seen from the calculations presented in Plate 6-9, the factor of safety against basal heave failure is in the range of 1.1 to 1.2. Under these conditions, the walls of the excavation would be expected to deflect more than 1½ inches, which could adversely impact the foundations of the Pacific Gateway Building.

Plate 6-10 shows an alternative approach to the construction of the station. The excavation would be divided in two cells. The western cell would be 100 feet wide, and the eastern cell, the one closest to the building, would only be 50 feet wide. The intermediate SPTC wall would run only the length of the Pacific Gateway Building (about 300 feet).

The Stage I excavation would be completed first, and the portion of the station within the limits of the Stage I excavation would be constructed. After the substructure and backfill are completed, the Stage II excavation can begin. As can be seen from the calculations on Plate 6-10, staged excavation and the presence of the intermediate wall would improve base stability, and reduce ground deformations.

The location of the intermediate wall can be adjusted in the lateral direction as required to improve constructibility of the substructures. It should be noted that on the west side of this excavation is a six-story masonry structure, at 301 Mission Street. We have no information on the foundations of this building, but it is very likely that the building is supported on timber piles that derive tip support in the dense Upper Sands. Depending on the actual condition of the foundations of the 301 Mission Street building, it may be necessary to install another intermediate wall near the west side of the excavation. In that case, the excavation would be subdivided in three cells rather than two. The middle cell would be excavated first, followed by the side cells.

Plate 6-11 shows the proposed shoring system for the terminal, for alternatives 2B and 2C, near Howard Street. The conditions at this location require special consideration because of the potential impacts of the excavations on the foundations of the 301 Howard Street building.

As can be seen from Plate 6-11, the pile foundations terminate at elevation -62 feet, which is about 7 feet below the base of the excavation. However, the piles are supported in the very dense Upper Sands, which provide good bearing. Because of the proximity of the shoring walls to the building foundations and because of the granular characteristics of the pile bearing strata, soil-cement walls would pose much less risk to the building foundations than SPTC walls. However, the walls would have to extend a significant distance below the building foundations to anchor them against lateral movements. Heavy soldier piles such as W30 × 211 would be required to provide adequate stiffness and control lateral soil deformations and settlements. Because of the large width of the excavations it will be very difficult to provide sufficient wall system stiffness to reduce the risk of lateral deflections to acceptable levels. It is therefore proposed that the soil behind the excavation, under the pile tips, as well as portions of the soil inside the excavation, below subgrade, be solidified using suitable grouting methods. Inclined grouting can be used to grout below the pile tips.

As an alternative to the grouting proposed, a system could be used similar to that shown on Plate 6-10, which uses an intermediate wall to reduce the width of the excavation adjacent to the building. As was the case for the excavation near the Pacific Gateway Building, the intermediate wall would be required only along the length of the building, perhaps extending 25 feet north and south of the building frontage. The excavation would be carried out in stages. The staging would be arranged to suit best the construction of the permanent box structures.

6.1.3 Excavations for Alternative 3: Terminal at Transbay Transit Terminal

The excavations at the Transbay Terminal would be relatively shallow as compared to the excavations for the terminal alternative on Beale Street. Excavation depths would range from 30 to 40 feet, reflecting the variations in ground surface elevations. Typical cross-sections of the proposed excavations are shown on Plates 6-12 and 6-13, representing areas of relatively shallow and thick Bay Mud, respectively.

There are several concerns with the proposed excavations. First and foremost is the stability of the base of the excavation against hydrostatic uplift. As the calculations on Plate 6-13 show, the factor of safety is less than 1.0 even in the areas where the thickest Bay Mud is found below the excavation base. This problem can be alleviated by dewatering the Upper Sands to draw the water level in this stratum below the bottom of the Mud. However, dewatering could cause compression of the Bay Mud due to reduction of the buoyancy forces and, therefore, could cause substantial settlements around the excavations. These settlements would impact adjacent buildings, streets and utilities. Another option is to extend the cut-off walls to penetrate into the Old Bay Clay, as shown on Plates 6-12 and 6-13, to cut off underseepage. The hydrostatic pressures in the Upper Sands,

within the limits of the excavation, could be reduced to safe levels using relief wells; without producing the unwanted effects that dewatering would have if the walls did not penetrate deep enough to tie into the Old Bay Clay layer.

The walls of the excavations can be supported using tiebacks as shown on Plate 6-12 or internal struts as shown on Plate 6-13. Because of the significant width of the excavation, tiebacks would be more economical. However, where the Bay Mud is thick, and the Upper Sands are correspondingly thinner, the tiebacks may be fairly long, or they may have to be drilled at a very steep angle. Steeply inclined tiebacks would transfer significant vertical downward loads to the shoring walls, which is not desirable. The walls would have to resist the downward component of the tieback loads in friction, because the end-bearing capacity would be very small (the walls tip out in the Old Bay Clay).

6.2 EXCAVATIONS ALONG THE RAIL LINE ALIGNMENTS

The ground conditions exhibit significant variability along each of the alignments under consideration. The approach to excavations associated with each alignment is to subdivide the alignment into areas where similar ground conditions and excavation depths are encountered and present our recommendations for each of these characteristic conditions. Along Alignment 2 there are some areas where special designs for the excavation and shoring are required. They include: (1) the excavation under the Harrison Street bridge, (2) the excavation in front of the west anchorage of the Bay Bridge; and (3) the excavations along the Embarcadero where the available space precludes the use of conventional excavation techniques unless special arrangements could be made to shut down the MUNI line, which runs within a few feet of the excavation, for the duration of the excavation. The shoring requirements for these special conditions are reviewed in Section 6.3, below.

6.2.1 Alignments 2A, 2B, 2C, Cut-and-Cover

6.2.1.1 Excavations Along Beale Street

South of the Beale/Market Station, the excavations for these alternatives remain very deep all the way to Harrison Street. South of Harrison Street, the depth of the excavation changes over a distance of 1,100 feet, from 70 feet deep to less than 50 feet deep. The depth of excavation decreases to about 45 feet near Bryant Street, decreasing further to about 42 feet near Brannan Street. It remains essentially at the same elevation all the way to the portal near Fifth Street. Between the portal and Sixth Street, the depth of the excavation decreases rapidly from 35 feet to zero, over a distance of about 800 feet.

South of the terminal, the ground conditions change dramatically between Howard and Folsom Streets. The Bay Mud pinches out and the depth to the top of the Old Bay Clay dips down to about 100 feet or so. At the same time, the depth to rock decreases dramatically. While at Howard Street the depth to rock is on the order of 200 feet, near Folsom Street it decreases to about 115 feet, and further south near Harrison Street the rock is found within a few feet below ground surface.

Plate 6-14 shows the subsurface section and excavation near Folsom Street, on the right-hand side, and a point about 200 feet north of Folsom on the left-hand side, for alternative 2A-C&C. Given the depth of the excavation, the walls should penetrate into the deep sands below the Old Bay Clay and the soldier piles should penetrate further into the rock at a depth of 150 feet. This design is essentially the same as the one proposed for the Beale/Market Street Terminal. As the depth to rock decreases, the walls could be terminated after they penetrate a few feet into the rock.

However, the vertical spacing of the struts could be increased as compared to that of the terminal excavation. Strut spacings between 12 feet and 14 feet are recommended. This would reduce the number of support levels from 8 to 6. The larger spacing should also improve the ease of excavation as there would be more headroom for excavating equipment to work under the struts.

Plate 6-15 shows the proposed shoring system for the area near Folsom Street, for alternatives 2B-C&C and 2C-C&C. Because the excavations are shallower than for alternative 2A-C&C, in-situ mixed soil-cement walls with steel soldier piles could be used rather than SPTC walls. The soldier piles would only need to extend to the tip of the soil-cement walls. The depth of the walls is dictated by the desire to achieve a positive cut-off against underseepage. This is a conservative recommendation, and it is recognized that further investigations during preliminary and final design may lead to further reductions in the depth of the walls. If the walls were terminated higher (above the Old Bay Clay and Bedrock), significant dewatering would have to be performed to control seepage into the excavation. The cost of dewatering, as well as the potential impacts of dewatering (which could cause settlements of adjacent structures), should be compared with the cost of the deeper walls shown in Plate 6-15.

One of the options under consideration for the alternative involving a station terminating at the intersection of Beale and Mission Streets is to provide a connection to the BART/MUNI Embarcadero Stations through an underground walkway. Plate 6-16 shows the cross-section for the proposed excavations and shoring system. As can be seen from Plate 6-16, the required excavations are 30 feet deep and 36 feet wide. The clear distance between the excavations and adjacent structures is about 22 feet. Under these circumstances, we recommend that the excavation walls be constructed

using in-situ mixed soil-cement walls and steel soldier piles, as indicated on Plate 6-16. The walls should extend through the Bay Mud to penetrate 10 feet into the Upper Sands. The lateral deformations associated with this excavation should be less than 1½ inches. Settlements should also be small and should have no impact on the adjacent structures because they are supported on piles that penetrate a significant distance below the bottom of the excavation.

South of Folsom Street the depth to the top of rock decreases rapidly. Rock at shallow depths (less than 25 feet) should be expected from a point about 250 feet north of Harrison Street all the way to Bryant Street. Along this segment of the alignment the excavations would be carried out in rock for a significant depth.

Plate 6-17 shows typical subsurface conditions and the proposed shoring system in the area between Harrison and Bryant Streets. The proposed shoring system would consist of steel soldier piles installed in predrilled holes at 6-foot spacings along the alignment. Because of the poor quality of the rock, powerful auger rigs should be used to drill a large percentage of the soldier pile holes. Special coring equipment would be used in the remaining holes where boulders or hard rock are found. The exposed rock between the soldier piles would have to be supported, as required, using wire mesh and shotcrete. Steel walers would be required to transfer the loads to the horizontal supports, which could consist either of tiebacks (rock anchors) or internal struts. The system of soldier beams and walers would form a lattice, which together with the wire mesh and shotcrete, would prevent massive rockfalls that could jeopardize the stability of the excavation. If rock anchors are used, they should be long enough to avoid failure of the mass in which they are embedded. Dewatering would be required. The volume of water that needs to be disposed would depend to a large degree on the nature of the rock. Sandstones may involve significant seepage into the excavations, while shales and siltstones are likely to result in smaller inflows of water.

6.2.1.2 Excavations Along the Embarcadero

Plate 6-18 shows the subsurface conditions and proposed shoring system along the segment of the alignment between Bryant and Brannan Streets, the point where the alignment meets the Embarcadero roadway. The ground conditions and shoring system shown on Plate 6-18 should be considered typical of what could be used also along the Embarcadero. The same system could be used for the portion of the alignment along Townsend Street, between the Embarcadero and a point about 600 feet west of the Embarcadero.

The walls of the excavations would be constructed using in-situ soil mixing and soldier piles, extending deep enough below the base of the excavations to tie into an impermeable clay stratum to

provide effective cut-off against underseepage. The walls are expected to range from 75 to 85 feet in depth in this area. The walls could be supported with three levels of struts as shown on Plate 6-18.

6.2.1.3 Excavations Along Townsend (King)

In the area between the Embarcadero roadway and a point about 200 feet west of First Street, the excavation shoring system would be essentially the same as the shoring system shown on Plate 6-18 for the area between Bryant and Brannan Streets (along Beale Street) and along the Embarcadero.

Farther west of First Street, the Bay Mud pinches out and the subsurface conditions consist of Fill over Older Bay deposits, a system of interbedded dense silty and clayey sands with occasional stiff clay lenses found between the Fill and bedrock. The excavations along this segment of the alignment are typically about 44 feet deep and about 45 feet wide. Plate 6-19 shows the proposed shoring system, consisting of in-situ mixed soil-cement and soldier pile wall and internal struts. Because the depth to rock varies substantially along the alignment, in some areas the walls would penetrate into the rock and could provide a reasonably effective cut-off against underseepage. In areas where the rock is deep, we propose that the walls be terminated at a depth of 65 feet below ground level, providing a toe embedment of 20 feet or so. Under these conditions, considerable seepage into the excavation may be encountered. Provisions for control of the water seeping into the excavation would be required. Lowering of the groundwater levels outside the excavation should be expected. By proper application of dewatering methods the area of influence of the groundwater lowering can be minimized. Because in this area the soils are quite dense, consisting primarily of granular soils and stiff silts and clays, settlements due to dewatering would be small.

In the area between Third and Fourth Streets the ground conditions are very similar to the subsurface conditions shown on Plate 6-19. We therefore recommend that the shoring system shown on Plate 6-19 also be used in the area between Third and Fourth Streets.

In the area between the intersection with Colin P. Kelly Jr. Street and Third Street, the rock is fairly shallow, and therefore the excavation would be in rock for considerable depth. Plate 6-20 presents our recommendations for shoring the excavations in this area. Rock anchors or internal struts could be used to support the excavation walls, depending on whether the necessary easements could be secured for construction of the rock anchors. The shoring system proposed for this area is very similar to that shown on Plate 6-17 for the area near Harrison Street. Depths of soldier pile embedment below excavation base would vary depending on rock quality and groundwater conditions.

6.2.2 Excavation Along Alternatives 3A/3B

Alternatives 3A and 3B involve cut-and-cover construction in the area between Folsom Street and the Transbay Transit Terminal, and along Colin P. Kelly Jr. Street from a point approximately 100 feet north of Brannan Street to Townsend Street. These alignments also include the proposed subway extension along Townsend Street from Fifth to Seventh Street. The excavations along these two alternatives are divided into three groups. The excavations north of Folsom Street, excavations between Brannan and Townsend Streets, and the excavations along Townsend Street west of Fifth Street. The excavations between Colin P. Kelly Jr. Street and Fifth Street along Townsend Street are the same as those discussed for alternative 2B.

6.2.2.1 Excavations North of Folsom Street

Plate 6-21 shows the proposed shoring system for the excavation near Howard Street. The shoring system consists of in-situ mixed soil-cement walls with steel soldier piles. Three levels of internal bracing as shown on Plate 6-21 will be required. The base of the excavation will be in the dense Upper Sands and, therefore, groundwater control is of concern. We recommend that the walls be extended to tie into the Old Bay Clay to provide a positive cut-off against underseepage into the excavation. Ground movements resulting from this excavation should be small because the soils supported by the excavation are generally stiff, with the exception of a thin layer of Bay Mud that is anticipated to have relatively little influence on the performance of the excavation.

6.2.2.2 Excavations Along Colin P. Kelly Jr. Street

Plate 6-22 shows the excavation and shoring system for the area near Brannan Street. This is the area of the tunnel access shaft, and the excavation would be about 70 feet deep. The subsurface conditions consist of fill over stiff clays over bedrock. The overburden soils are about 30 feet thick. The excavation extends approximately 40 feet into Franciscan rock.

The wall retaining the ground behind the excavation can be constructed as a tangent cylinder pile wall. The soldier piles will have to be installed in predrilled holes, and should extend 10 to 15 feet below excavation subgrade. Dewatering will be required to control seepage into the excavation. The amount of seepage entering the excavation would vary significantly depending on the type of rock exposed in the excavation.

Excavations along Townsend Street west of Third Street would be similar to the excavation and shoring system shown on Plate 6-19. Excavations along Colin P. Kelly Jr. Street between Brannan and Townsend Streets would be similar to the excavations shown on Plate 6-20.

6.2.2.3 Extension of Cut-and-Cover Alignment Between Fifth and Seventh Streets

In the area near Fifth Street and west of Fifth Street the thickness of the Bay Mud increases very significantly. Between Fifth and Sixth Streets, the depth to the bottom of the Bay Mud is about 100 feet. Similarly, the depth to rock increases dramatically from 120 feet near Fifth Street to about 180 feet near Sixth Street. Between Sixth and Seventh Streets the thickness of the Bay Mud decreases as well as the depth to the top of rock, reflecting the fact that this area is closer to the old shoreline where water depths were shallow and deposition of soft sediments was less prevalent.

Plate 6-23 shows the subsurface conditions and proposed shoring at Fifth Street, where the Bay Mud extends to a depth of 100 feet. The proposed shoring system consists of in-situ mixed soil-cement walls with steel soldier piles every 4 feet. The wall should penetrate to a depth of 65 feet, but every second soldier pile should be driven to penetrate into the more competent soils (dense sand layers or Old Bay Clay) below the Bay Mud. Secondary piles should penetrate 20 feet below the tip of the soil-cement wall.

Plate 6-24 shows the proposed shoring for the area near Sixth Street. The shoring system is very similar to the system at Fifth Street, shown on Plate 6-23. The main difference is that the soldier piles are terminated in a dense sand layer at a depth of 80 feet.

Plate 6-25 shows the subsurface conditions and shoring system near Seventh and Berry Streets, where the alignment turns toward the south. Because the Bay Mud is not as deep at this location as near Fifth and Sixth Streets, the excavation walls could be terminated in the rock at a depth of 55 feet or so.

6.3 SPECIAL CONDITIONS

In certain areas of the cut-and-cover alignments, particularly for Alternative 2, the potential impacts on adjacent structures and space availability to perform the work require special consideration of certain aspects of the excavation process, the shoring system, and the impacts that ground movements associated with the excavations would have on adjacent structures.

This section addresses the situations prevailing in the following three areas:

- Along the Embarcadero roadway between Brannan and Townsend Streets, where access is very limited because on the one side the work space is constrained by existing buildings and on the other side by the MUNI tracks along the median.
- Along Beale Street, in front of the Bay Bridge anchorage. The excavations at this location for alternative 2A-C&C will be approximately 75 feet wide and 55 feet deep. The base of the excavation will extend below the base of the Bay Bridge anchorage. Special construction procedures and sequencing will be required to avoid the undesirable impacts of ground movements associated with conventional excavation techniques on the bridge anchorage structure. The excavations for alternatives 2B-C&C and 2C-C&C would be shallower and narrower, but their impacts on the bridge foundations must also be evaluated
- Along Beale Street, under the Harrison Street Bridge. For alternative 2A-C&C, the two sides of the excavation will be right against the spread footings supporting the bridge piers. The excavation, which would extend 55 feet below ground level, will expose the bridge foundations to settlements and lateral movements that may be beyond the tolerance of the structure. Special procedures will have to be employed to protect the bridge foundations.

6.3.1 Excavations Along the Embarcadero Roadway

Plate 6-26 shows the subsurface section and the outline of the proposed excavation relative to the existing structures and facilities, such as the residential buildings along the southbound lanes, the underground box sewer structure, and the MUNI railroad tracks and overhead electrification lines. It may be seen from this plate that there is very little room on either side of the excavation and it would be very difficult to operate heavy construction equipment without the risk of accidental damage either to the buildings or to the railroad line. In addition, conventional excavation techniques would cause settlements and lateral deformations that would impact the existing buildings and the box sewer.

One of the options that could be employed to alleviate some of the difficulties associated with this congested site is to use tunneling methods to construct the rail line. However, even tunneling methods present significant challenges because of the limited space available within the right-of-way. As can be seen from Plate 6-27, the clear space between the tunnels is only 12 feet, which is one-half

of the tunnel diameter. Experience with soft ground tunneling techniques suggests that the clear space between adjacent tunnels should not be less than 3/4 of a tunnel diameter, and should preferably be equal to one tunnel diameter (i.e., 24 feet).

To alleviate the problems with the inadequate clear distance between the tunnels, we propose that the ground be treated by jet-grouting within the limits of the area shown on Plate 6-27. The Bay Mud soil would be mixed with a jet of cement grout to form a material that would be many times stronger than the Bay Mud. This technique was recently used to construct two tunnels for the Clean Water Program of the City of San Francisco in the Islais Creek area. The tunnels were 13½ and 15 feet in diameter, respectively, and were constructed in Soft Bay Mud. It is reported that the method was very successful in limiting settlements under a railroad embankment within tolerable limits (World Tunneling, June 1995). The treated Bay Mud material had strengths in the range of 600 to 1,000 psi, according to the same report.

A roadheader with a circular tunnel shield would be used to construct the tunnels. The shield would be advanced by jacking against the permanent tunnel lining that would be erected behind the tunnel face in the shield.

Other tunneling methods that have been considered for this portion of the alignment include mining under compressed air in an open shield and the use of an Earth Pressure Balance (EPB) Shield to construct the tunnels.

The use of compressed air under mixed-face conditions, involving fill over the Bay Mud, presents considerable risk due to loss of air from the face that could migrate and potentially invade private property, including the residential buildings in this area. The air could transport with it contaminants that are known to exist within portions of the waterfront fills and Bay Mud, such as hydrocarbons, methane and hydrogen sulfide. It must also be realized that significant settlements could develop during tunneling and there is still a serious risk of cave-in due to loss of stability at the tunnel heading. It must also be recognized that productivity is low (on the recently constructed MUNI tunnels at the foot of Market Street, progress was 15 feet per three 8-hour shifts) and the mobilization and maintenance costs, which include significant health and safety–related costs, are very high.

The use of an earth pressure balance (EPB) shield for tunneling in soft ground is very attractive. This method has been used for the construction of the N1/N2 tunnels for the north outfall consolidation project in the Marina and North Beach areas of the City. The experience from use of the EPB shield was that significant settlements can develop. However, more important is the risk that

the machine can get stuck if it encounters obstructions such as a significant number of piles or ships that may be buried in the fills and Bay Mud along the waterfront. If obstructions were to be encountered, the only way to free the machine and continue tunneling would to be excavate a shaft in front of the shield to gain access and remove the obstruction. Such an undertaking would not only be very expensive and cause delays to the project but would also cause very significant disruption to the traffic and nearby residences. Because the timing and number of such occurrences cannot be predicted, plans cannot be made to deal effectively with such occurrences.

It must also be recognized that the use of EPB shields is very expensive. Considering the costs and risks associated with the use of this method, it was agreed after consultation with the Board of Consultants for this project that the use of EPB machines involves substantial risks with respect to the presence of subsurface obstructions, and that it would be advantageous to pursue alternate excavation methods at this stage.

An alternative excavation method to the tunneling proposal is illustrated on Plates 6-28a and 6-28b. The excavation would be constructed in stages. First, the excavation walls would be installed using in-situ mixed soil-cement and soldier piles. After installation of the walls, the soils between the walls from 20 feet to 55 feet in depth would be treated by jet-grouting, to create a preformed strut between the walls. At this stage it is envisioned that the struts could be 10 feet wide and installed 20 feet apart, to minimize the amount of jet grouting. The next step is to excavate between the walls to a depth of 6 feet or so, install and preload the first level of struts, and construct a road deck supported on the excavation walls. Once the deck is completed, the roadway would be opened to traffic, and excavation would continue from both ends of this segment below the deck. The various subsequent steps in the excavation process are illustrated on Plate 6-28b. A variation of this alternative is to delay jet grouting until the first level of struts and decking are in place. Jet grouting could be performed from the deck, working at night, and the spoils of jet grouting could collect in the trench below the deck, thus minimizing the effects of jet grout spoils on traffic. This would be from an environmental point of view a better alternative, and is referred to as the environmentally sensitive cut-and-cover (ESCC) alternative.

During excavation, the jet grouted soil would act as a very stiff strut holding the walls in place and limiting deflections to very small values. The walls would behave as stiff beams fixed at the base and simply supported at the top, with the intermediate strut preventing excessive beam deflections. It is our assessment that this procedure could control lateral deflections and settlements to values less than one inch. Of course, the greatest advantage of this method is that during excavation it avoids interference with the buildings, the traffic, and the MUNI railroad line. However, during installation

of the walls, during jet grouting, and until the deck is constructed, interference with traffic should be anticipated, although it could be minimized by careful planning.

6.3.2 Excavation Under the Harrison Street Bridge

The outline of the excavation relative to the foundations of the Harrison Street Bridge is shown on Plate 6-29. As can be seen on that plate, the construction of the excavation would impact the foundations of the bridge. Portions of the footings may have to be removed to facilitate the excavation. During the excavation process the stability of the footings against overturning may be affected, unless the footings are underpinned.

The walls would be constructed by underpinning methods, as a series of hand-excavated piers, spaced 5 feet apart. The piers would be advanced 5 to 10 feet below the general excavation level at each successive step. The method of construction would involve advancing one pier or several piers simultaneously, but small enough in number, and located in such a manner as not to affect the existing foundations. The piers would be used to support the footings as the excavation progresses.

In addition to the underpinning provided by the excavation wall, inclined rock anchors should be used, if required, to provide lateral restraint of the piers away from the excavation as shown on Plate 6-29. In addition, grouting should be carried out under the footing to strengthen the rock behind the excavation to avoid potential settlements during construction of the wall. Closely spaced rock anchors (10-foot vertical and horizontal spacing) should be used and preloaded to control lateral deflections and restore lateral ground stresses under the footings.

One of the alternatives for the cut-and-cover alignment would involve a two-track system that would involve a much narrower and shallower excavation than shown on Plate 6-29. The excavation would be 40 feet wide and 42 feet deep, as shown on Plate 6-30. The excavation walls are about 20 feet away from the bridge piers.

Excavation influence lines drawn at slopes of 1.0 H to 2.0 V and at 1.0 H to 1.0 V are shown to illustrate the zones that may be affected by the proposed excavations. The influence lines shown on Plate 6-30 are usually used as guidelines for the development of underpinning and other support requirements of adjacent structures, for excavations in soil. The use of these influence lines for excavations in rock is conservative but is justified at this stage of the project, because of the obvious importance of the Harrison Bridge and because the rock may be of very poor quality and may behave like soil.

Typically, foundations that derive their support below the 1.0 H to 2.0 V line do not require underpinning, but would be impacted by the excavations. Foundations that derive their support below the 1.0 H to 1.0 V line do not require underpinning and the impact of excavations on adjacent structures is usually within tolerances.

In view of the fact that portions of the bridge footings are within the 1.0 H to 2.0 V zone (even if it is a relatively small portion of the foundations), measures must be taken to protect the bridge foundations. We recommend that the rock behind the excavation, and especially under the footings, be grouted to make it possible to transfer the loads deep below the 1.0 H to 1.0 V influence line. The required amount of grouting will have to be evaluated during the detail design phase of the project. If the rock is of sufficiently good quality, grouting may not be necessary.

The shoring for the excavation could consist of soldier piles spaced 5 feet apart, penetrating about 10 feet below the base of the excavation. Walers and struts could be used to support the walls. The soldier piles and walers could form a lattice, preventing rock instability behind the walls. The walls would be treated with wire mesh and shotcrete to provide further support and act as lagging, especially in areas where the exposed rock is of poor quality.

6.3.3 Excavations at West Bay Bridge Anchorage

6.3.3.1 Seventy-five-foot-wide Excavations

Plate 6-31 shows the cross-section of the excavation in front of the west Bay Bridge anchorage. It is 75 feet wide and 55 feet deep and the west wall of the excavation is within two feet of the face of the anchorage block. At full depth, the base of the excavation is more than 30 feet below the deepest part of the anchorage block.

Conventional excavation procedures would involve construction of vertical walls and use of tiebacks (rock anchors) or internal bracing (struts) to support the excavation. Details of the anchorage block design are not available; therefore, at this stage a detailed analysis of the effects of a conventional excavation on the anchorage block is not possible. Although it is feasible by proper sequencing of the excavation and by proper bracing to restore the lateral resistance that is currently provided by the volume of rock planned for excavation, it is recognized that use of conventional excavation techniques could cause unacceptable ground movements in terms of the performance of the anchorage.

In view of the potential for ground movements during excavation, alternative excavation procedures are examined below, with the objective of controlling ground movements to insignificant levels during excavation.

Regardless of what excavation scheme is used, it is considered essential that a fairly stiff wall be constructed to retain the rock under the anchorage. The wall can be constructed using concrete cylinder piles to form a tangent wall with heavy steel soldier piles for reinforcement. Special drilling equipment may be required to construct the cylinder piles. Equipment such as reverse circulation drilling (RCD) machines, that can operate under limited headroom, could be used, although the mobilization and operation of these machines is very costly. RCD machines are not readily available in the United States, but they are being used extensively in Europe and Asia to construct foundations of major high-rise buildings. We have had experience with this method, and we consider the use of this method technically feasible for this application. To minimize costs and expedite construction schedule, we propose that powerful auger drilling equipment be used to start and advance the holes as deep as possible. The RCD machines would complete the work after the auger drilling equipment has reached a point where the rock is too hard to be excavated with augers.

Once the walls of the excavation have been constructed, excavation can commence in stages. At each stage, only a small volume of rock would be removed, the walls of the excavation would be secured with struts and then the excavation process would continue.

Plates 6-32a, 6-32b and 6-32c show a staged excavation plan that would allow the excavation to be completed with minimal impact on the anchorage. The first stage involves the construction of an excavation 38 feet wide and 30 feet deep in the central portion of the final excavation. In the longitudinal direction the excavation would be opened up in short segments, probably 25 to 30 feet wide. In view of the small volume of material excavated at each stage, the internal walls at State I may not be required. This should be investigated further during the preliminary engineering phase.

After the Stage I excavation is completed, a 7-foot-thick slab would be constructed to form the roof of the central box of the CalTrain box structure. The space over the slab would be backfilled with lean concrete mix.

The second stage of the excavation would involve excavation of two narrow boxes, one on each side of the portion excavated during Stage I. The slabs forming the roof of the two side boxes of the CalTrain box structure would be constructed and the excavated hole backfilled with lean concrete mix.

Once the roof of the box structure has been completed, the remaining box structure would be constructed by mining techniques. The roof of the excavation walls would provide protection from potential difficulties in case poor rock is encountered in the excavation. The sequence of mining is illustrated in Plate 6-32c.

Another option is illustrated in Plates 6-33a and 6-33b. The basic assumption is that the excavation could be carried out in short longitudinal segments, over the full width of the excavation. Segments 25 to 30 feet long could be excavated and braced as shown. Then the slab forming the roof of the permanent box structure would be constructed within each segment and lean concrete mix backfill placed over the slab to fill the excavation. The main advantage of this method is that the slab could be constructed monolithically in the transverse direction, which would provide a stronger roof over the space that would be mined in Stage II. Once the slab has been constructed over the entire length of the anchorage face, the rock would be excavated under the roof of the permanent structure using mining techniques. The excavation would be carried out in substages, as shown on Plate 6-33b.

The basic premise for the success of the methods described above is that by removing relatively small sections of rock, at any one time, the impact on the foundation of the anchorage would be insignificant.

The final method for the 75-foot-wide excavations that would probably cause the least impact on the anchorage is illustrated on Plate 6-34. Microtunneling methods can be used to construct a series of small diameter tunnels, between the side walls, that are backfilled with concrete to form an enclosure encompassing the area of the permanent box structure. The excavation is then carried out using tunneling techniques within the confines of the arches formed by the previously constructed drifts.

6.3.3.2 Forty-foot-wide Excavations

One of the alternatives being considered would involve a two-track line rather than the original plan that would involve a four-track rail system. The advantage of the two-track rail system is that the required excavations would be considerably shallower and narrower than required for the four-track system.

Plate 6-35 shows the arrangement and configuration of the proposed excavation relative to the existing bridge anchorage. The west wall of the excavation would be 32 feet away from the face of the anchorage.

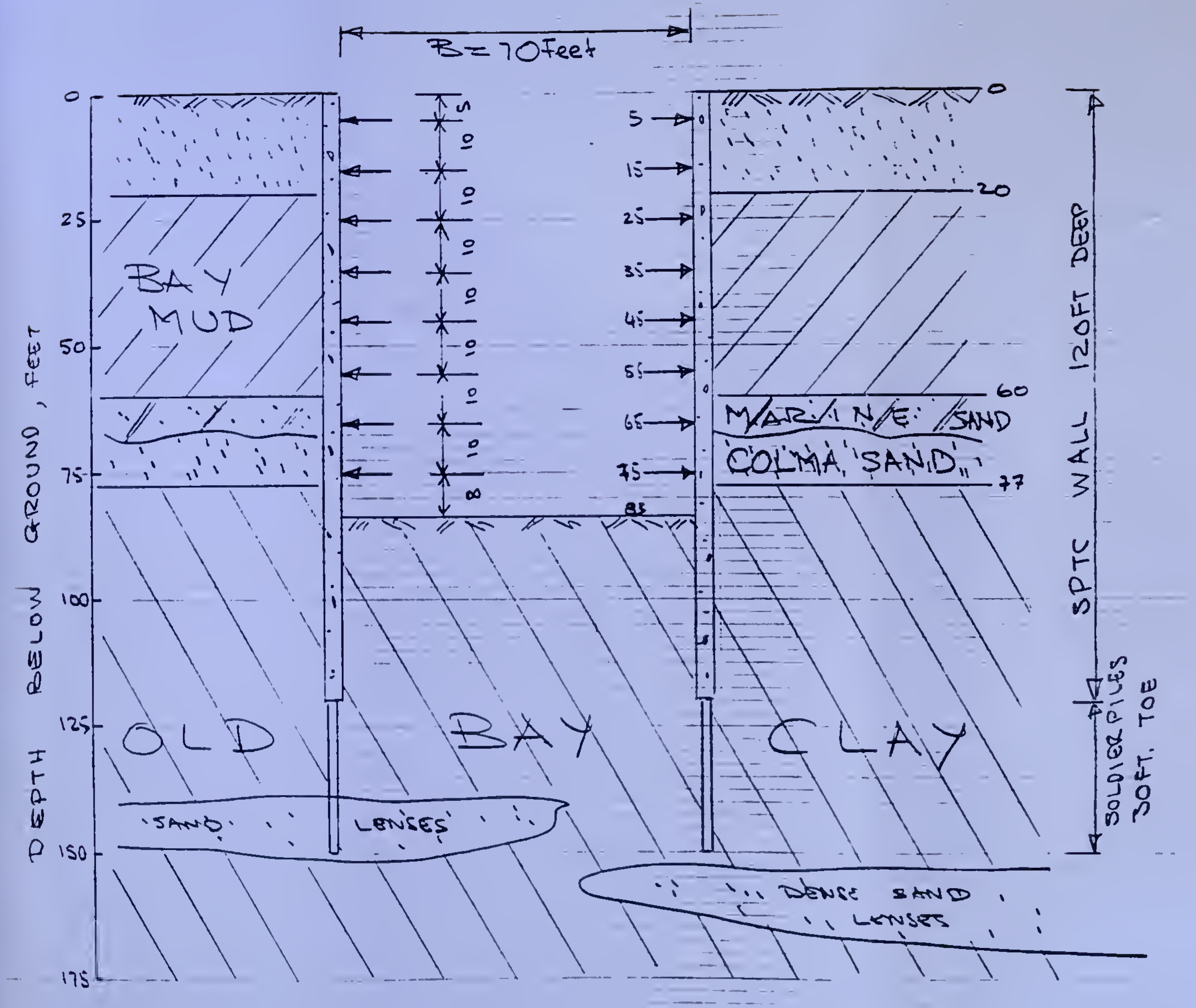
The revised plan and profile make it possible to construct the excavation using conventional shoring methods. We recommend that the walls be constructed as a system of steel soldier piles, spaced 6 feet apart, with welded wire mesh and shotcrete used to support the exposed rock between the soldier piles and walers. The use of walers and struts to support the excavation walls is preferred, rather than rock anchors. The combination of soldier piles and walers is intended to form a lattice support system to minimize the potential for rockfalls.

Because of the smaller depth of excavation and because of the significant distance between the excavation and the bridge anchorage (32 feet), we conclude that the excavation should have no impact on the Bay Bridge anchorage. On Plate 6-35, excavation influence lines are shown. These influence lines are used as criteria for underpinning requirements for excavations in *soil*. Therefore, application of these influence lines in rock is considered very conservative but appropriate in view of the obvious importance of the Bay Bridge anchorage.

Zone A, shown on Plate 6-35, which involves the soil behind the excavation within an imaginary line sloped at 1.0 H to 2.0 V, defines the zone where the impacts of the excavation on existing structures would be severe enough to require protection of the structures by underpinning. In Zone B, which is between the imaginary lines sloping at 1.0 H to 2.0 V and 1.0 H to 1.0 V, defines the limits where existing structures whose foundations are supported in this zone may be impacted but do not normally require underpinning. Zone C, which is located beyond the hypothetical line sloping at 1.0 H to 1.0 V, defines the area that is unaffected by excavation.

As can be seen from Plate 6-35, only a small portion of the Bay Bridge foundation area is inside Zone B. Given the conservative nature of these criteria when applied to excavations in rock and considering the shoring system that is recommended, it is concluded that the proposed excavations would have no effect on the west Bay Bridge anchorage.

Even though the planned excavations should have no impact on the Bay Bridge anchorage, we recommend that a monitoring program be implemented to verify this assessment. In the unlikely event that there is any indication that the excavation could impact the Bridge anchorage, the excavation procedures could be changed to protect the anchorage. Appropriate measures might involve excavation in short sections (25 feet long) in the longitudinal direction so that very little rock volume is removed in front of the anchorage at any given time.



NOTES:

1. THE SHORING SYSTEM CONSISTS OF: 3FT THICK CONCRETE WALLS WITH STEEL SOLDIER PILES
2. SOLDIER PILES EVERY 6FT. CONSISTING OF W36×300 SECTIONS
3. PRIMARY SOLDIER PILES 150FT LONG. SECONDARY PILES 120FT.
4. INTERNAL STRUTS AS SHOWN

EXCAVATION AND BRACING SYSTEM, BEALE STREET STATION EXCVATIONS: MARKET TO HOWARD STREETS

December 1995
30751-001-003

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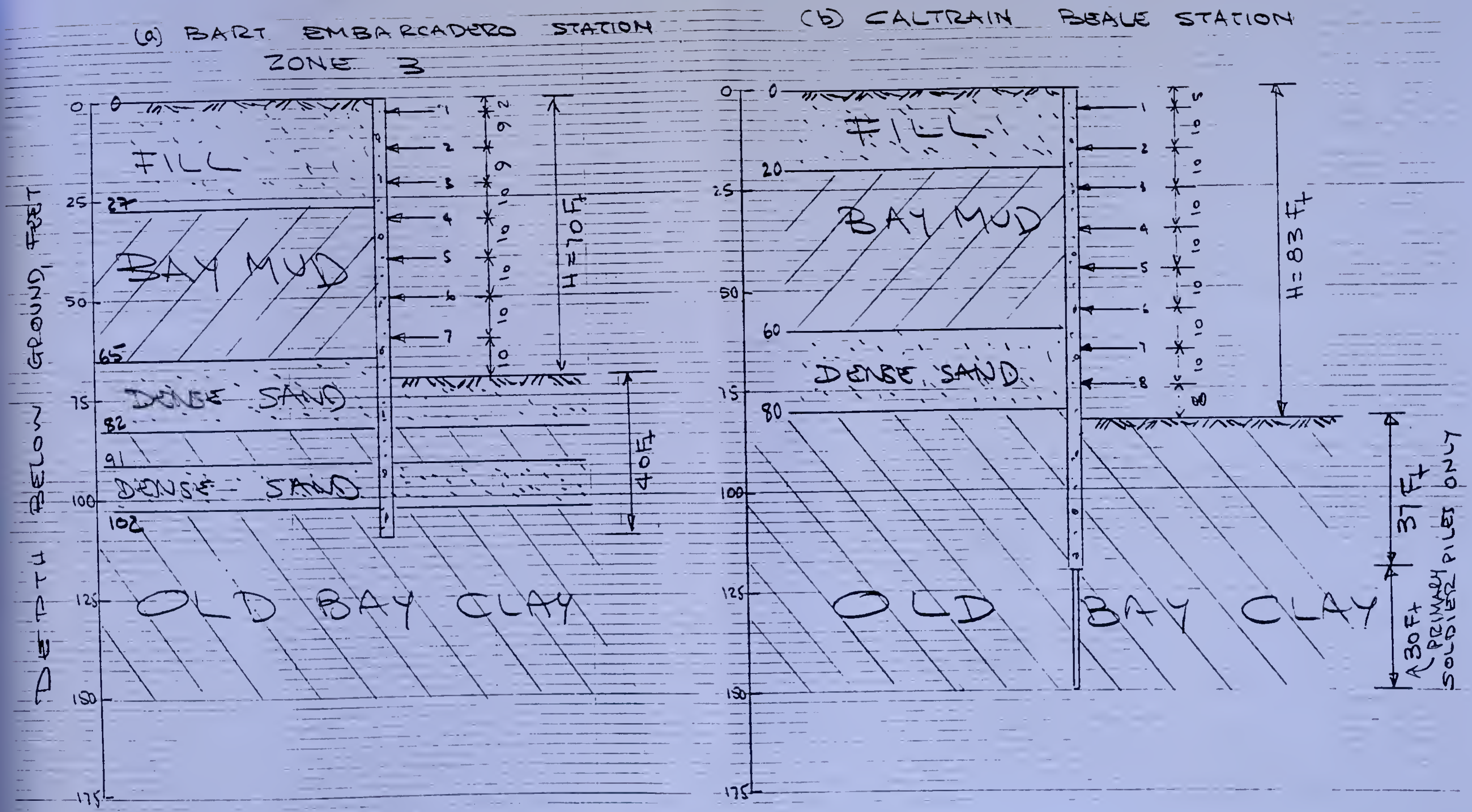
Caltrain S.F. Downtown Station
Relocation EIS/EIR Project

DAMES & MOORE

PLATE 6-1



COMPARISON OF SHORING SYSTEMS: BART AND CALTRAIN STATIONS

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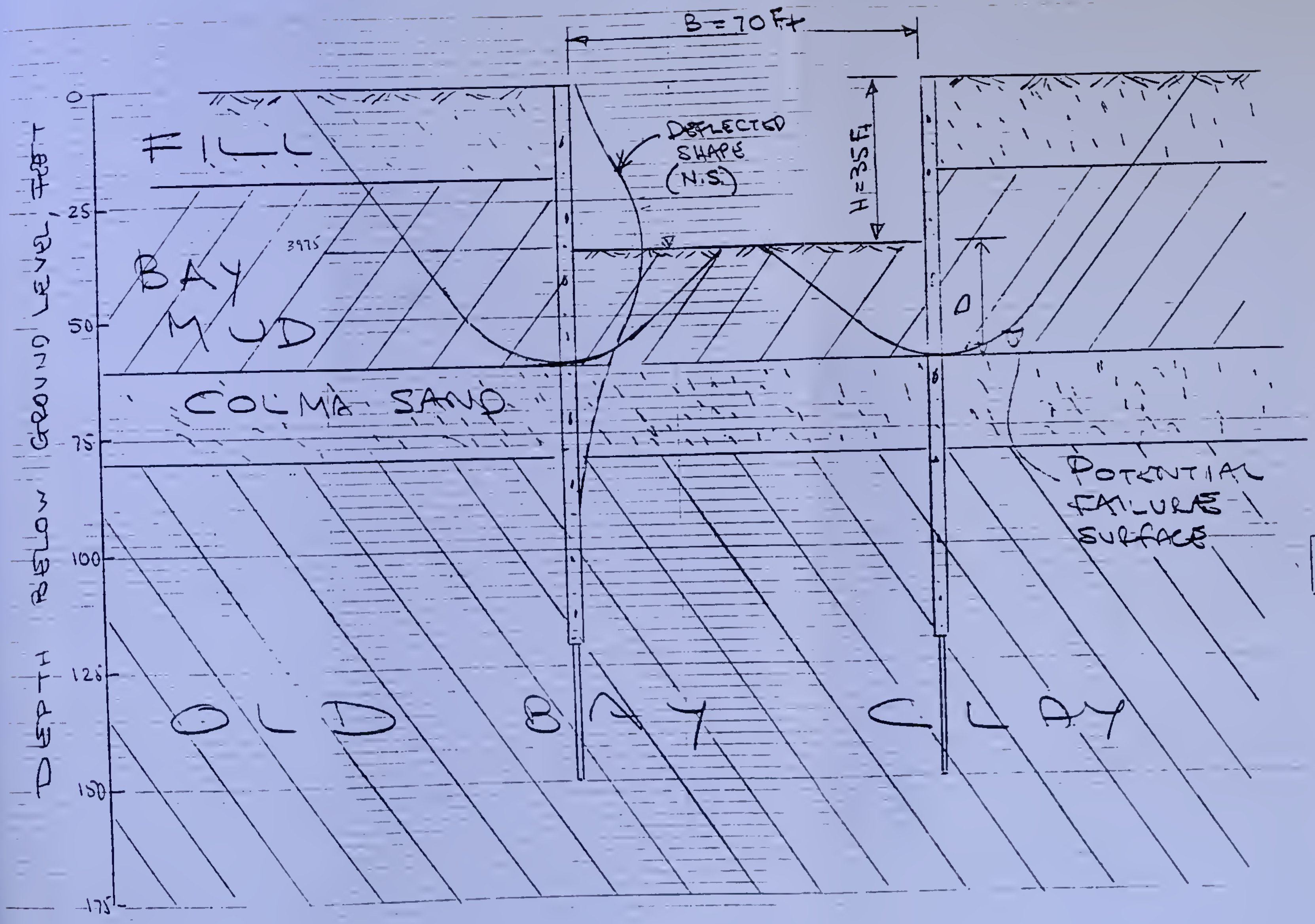
PLATE 6-2



BASE STABILITY

$$F.S. = \frac{c\,N_c + (S_u D/B)}{\gamma H + q}$$

$c = 900$ PSF

$D = 25$ Ft

$B = 70$ Ft

$N_c = 5.7$

$\gamma H = 4000$ psf

$q = 200$ psf

$\boxed{F.S = 1.3}$

EVALUATION OF BASE STABILITY -
35 FOOT DEEP EXCAVATION:
BEALE/MARKET STREET STATION

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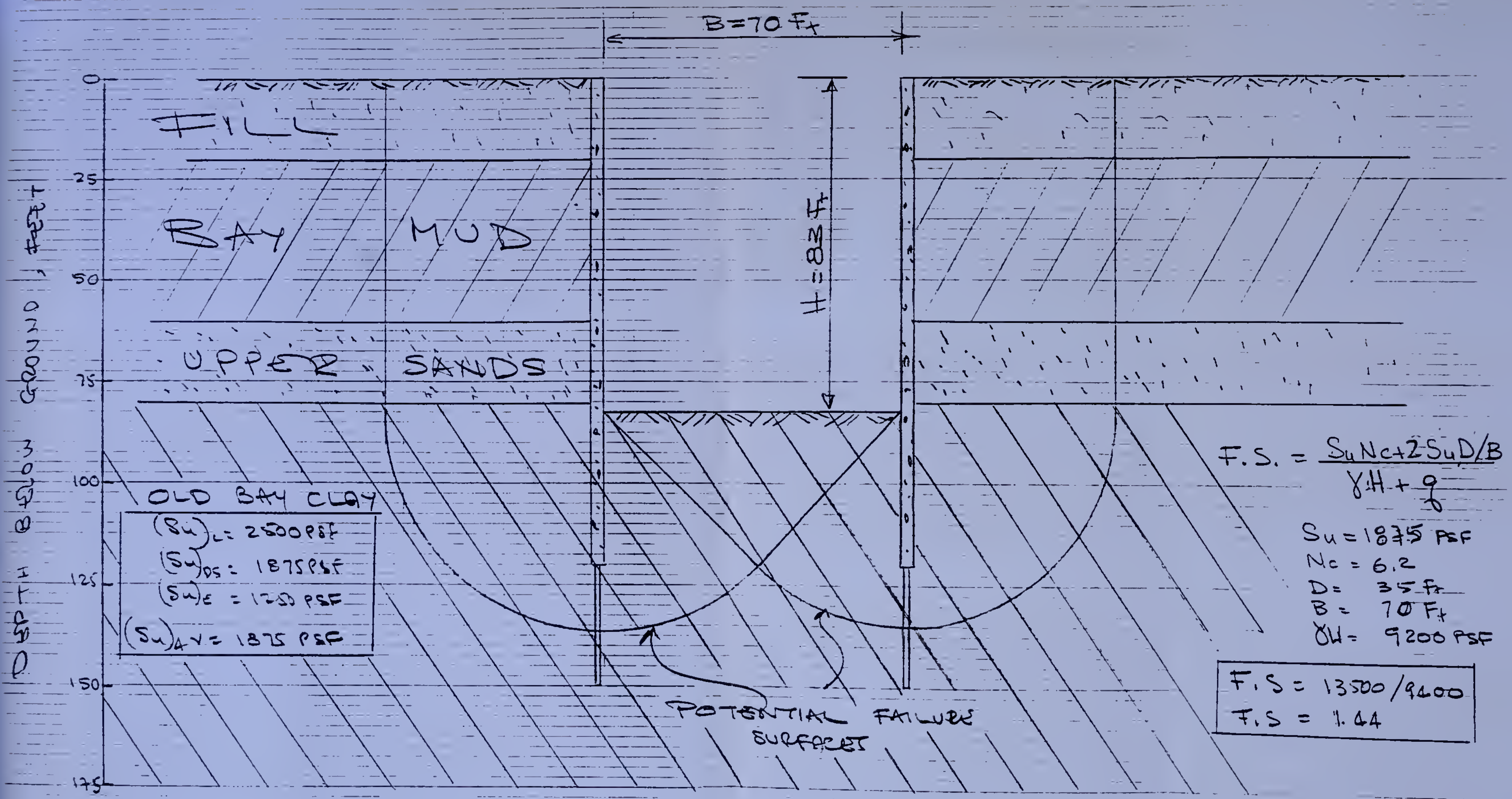
PLATE 6-3



EVALUATION OF BASE STABILITY -
FINAL EXCAVATION DEPTH:
MARKET/BEALE STREET STATION
ALTERNATIVE 2A-C&C

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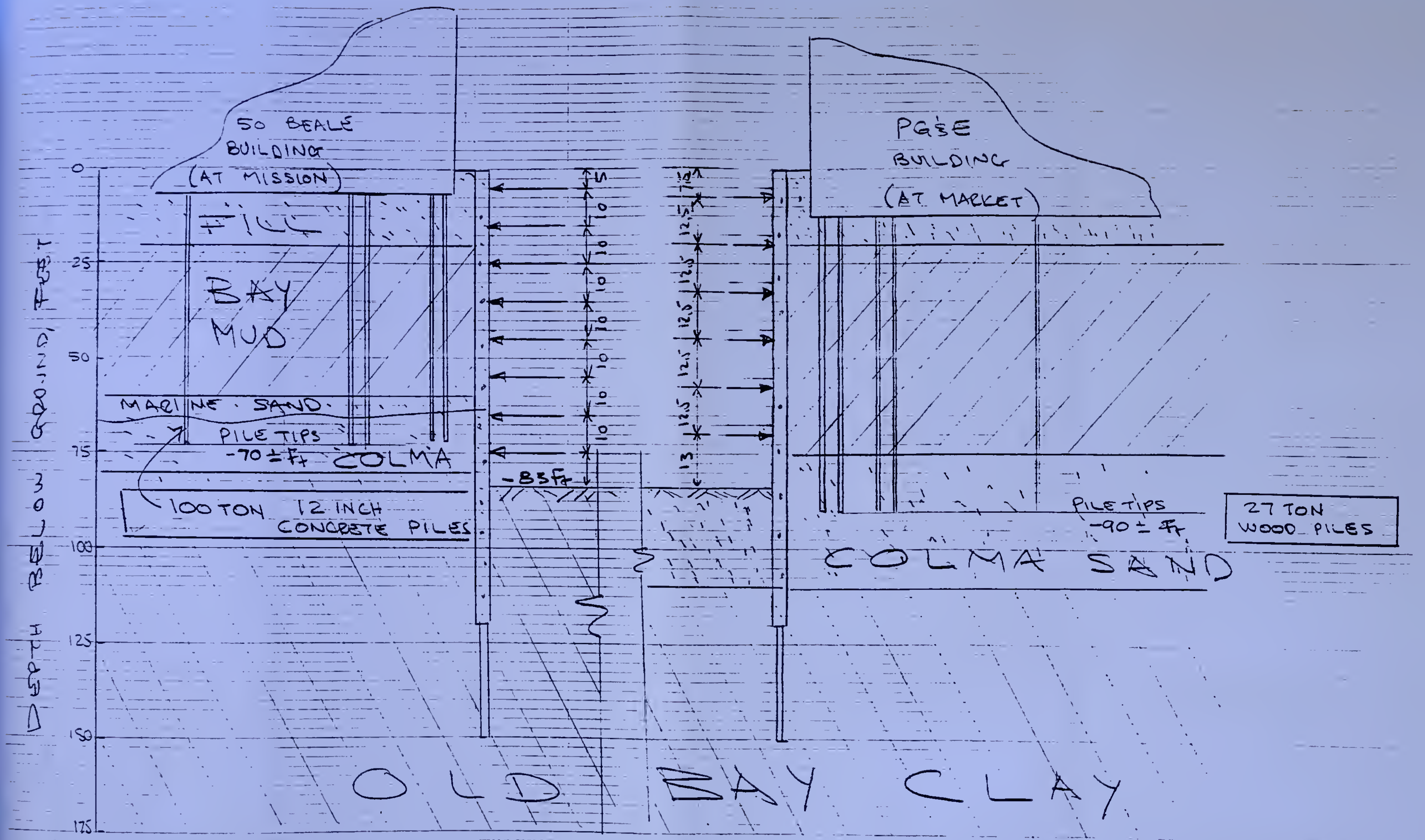
PLATE 6-4



RELATIVE POSITIONS OF ADJACENT BUILDING FOUNDATIONS TO THE BEALE/MARKET STREET EXCAVATIONS

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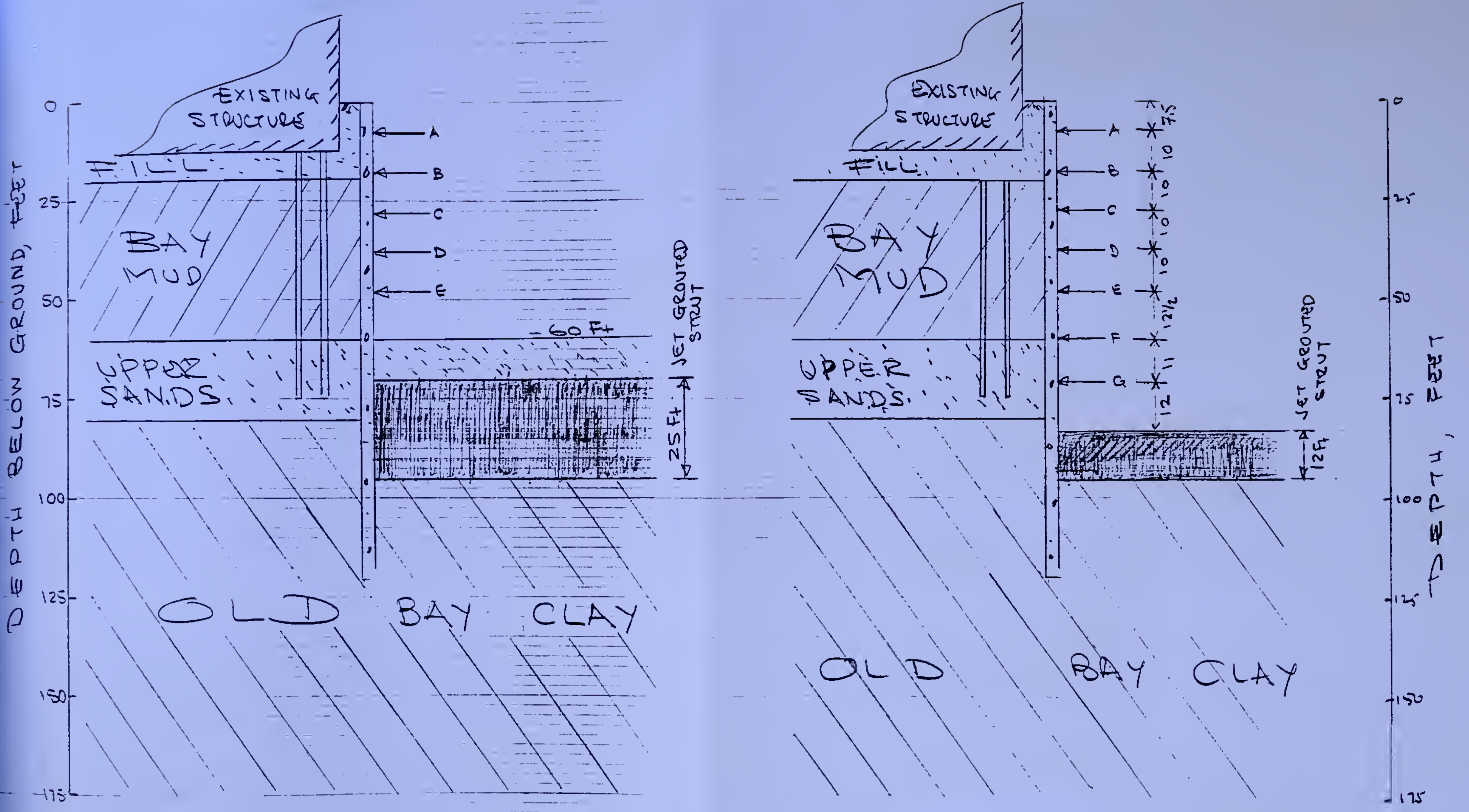
PLATE 6-5



(a) STAGE I EXCAVATION

(b) FINAL EXCAVATION STAGE

CONSTRUCTION OF A JET-GROUTED SLAB TO CONTROL DEFLECTIONS OF THE SHORING SYSTEM AND SETTLEMENTS ALTERNATIVE 2A-C&C

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Relocation EIS/EIR Project

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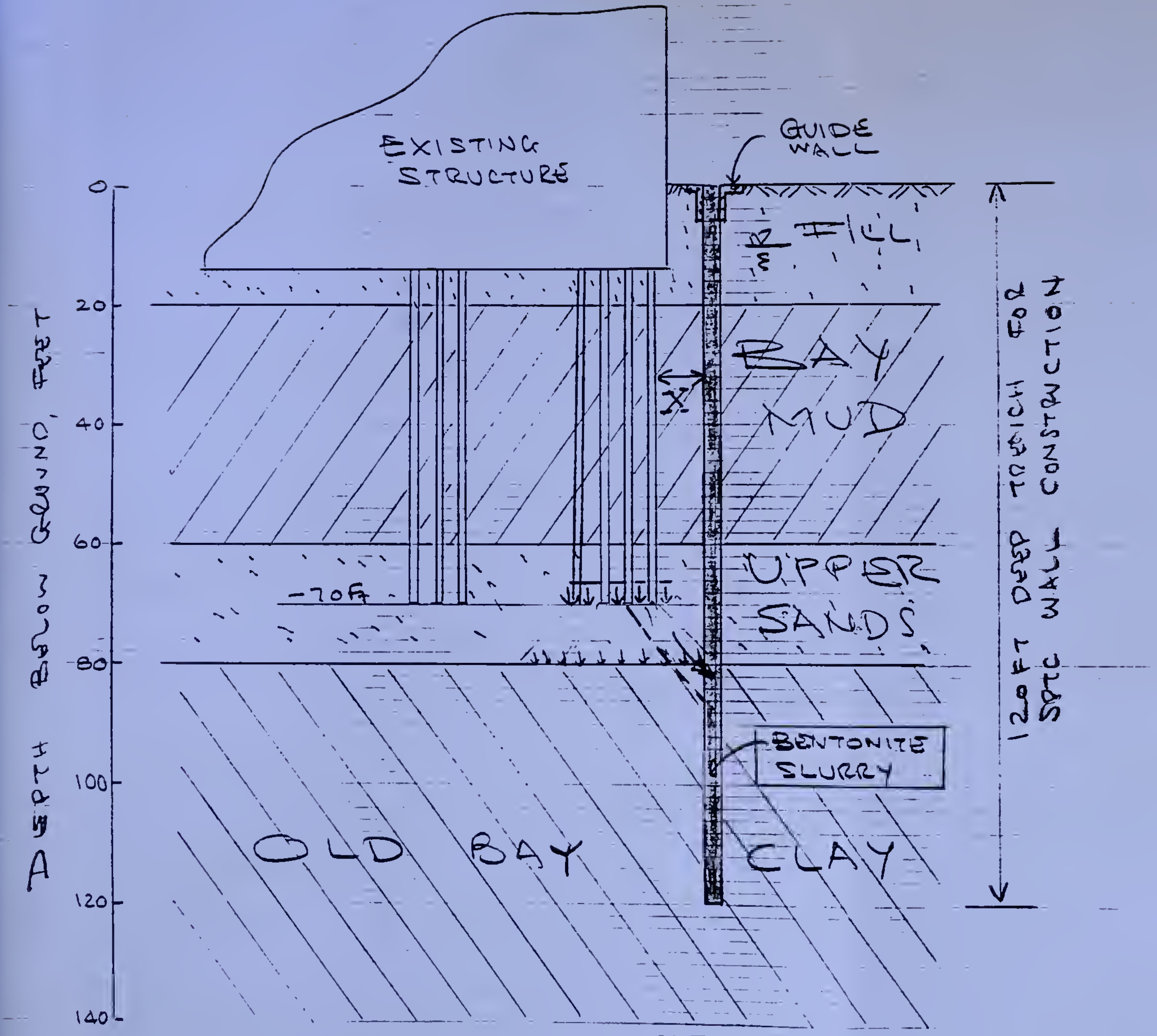
PLATE 6-6



NOTES:

1. DISTANCE BETWEEN EXISTING PILES AND SLURRY TRENCH MAY BE 5-10 FEET
2. TRENCH EXCAVATION MAY CAUSE RELIEF OF HORIZONTAL STRESSES AT FOUNDATION LEVEL CAUSING PILE SETTLEMENTS
3. ACCIDENTAL TRENCH COLLAPSE MUST BE AVOIDED TO PROTECT THE PILES, TO ACCOMPLISH THIS IT MAY BE NECESSARY TO JET GROUT THE SAND LAYER, BEFORE TRENCH EXCAVATION BEGINS

POTENTIAL FOR UNDERMINING EXISTING PILE FOUNDATIONS DURING SLURRY TRENCH EXCAVATION

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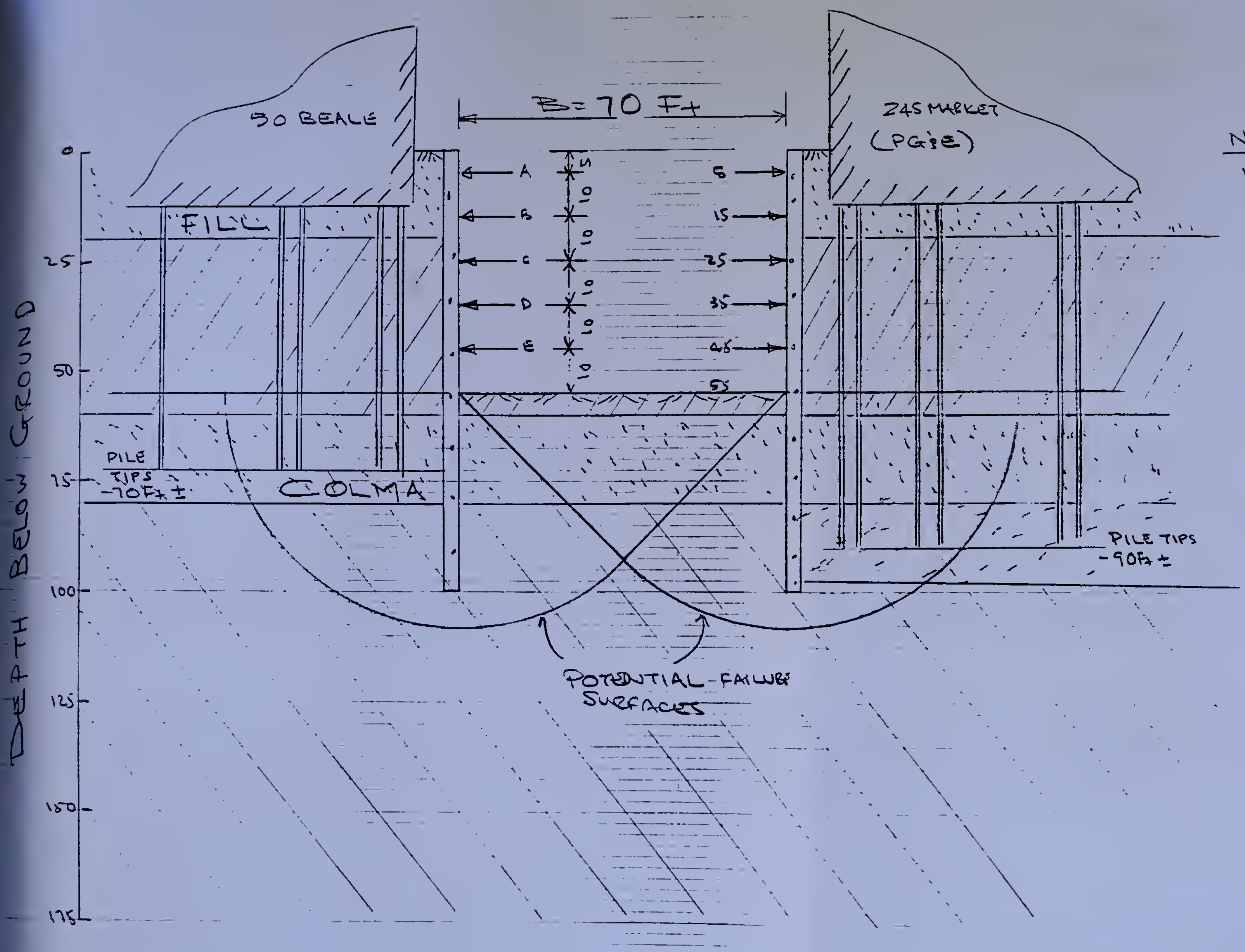
PLATE 6-7



NOTES

1. SHORING SYSTEM CONSISTS OF SOIL-CEMENT, IN-SITU MIXED WALL, 3FT THICK
2. SOLDIER PILES CONSIST OF W30X211 STEEL SECTIONS
3. SOLDIER PILES SPACED 4FT CENTER TO CENTER, 100FT LONG
4. JET-GROUTED SLAB (STRUT) 20FT THICK BELOW EXCAVATION SUBGRADE TO CONTROL DEFLECTIONS BELOW EXCAVATION BASE
5. INTERNAL STRUTS AS SHOWN LONGITUDINAL SPACING 15 FT

EXCAVATION BRACING SYSTEM FOR THE BEALE/MARKET STREET STATION ALTERNATIVE 2B

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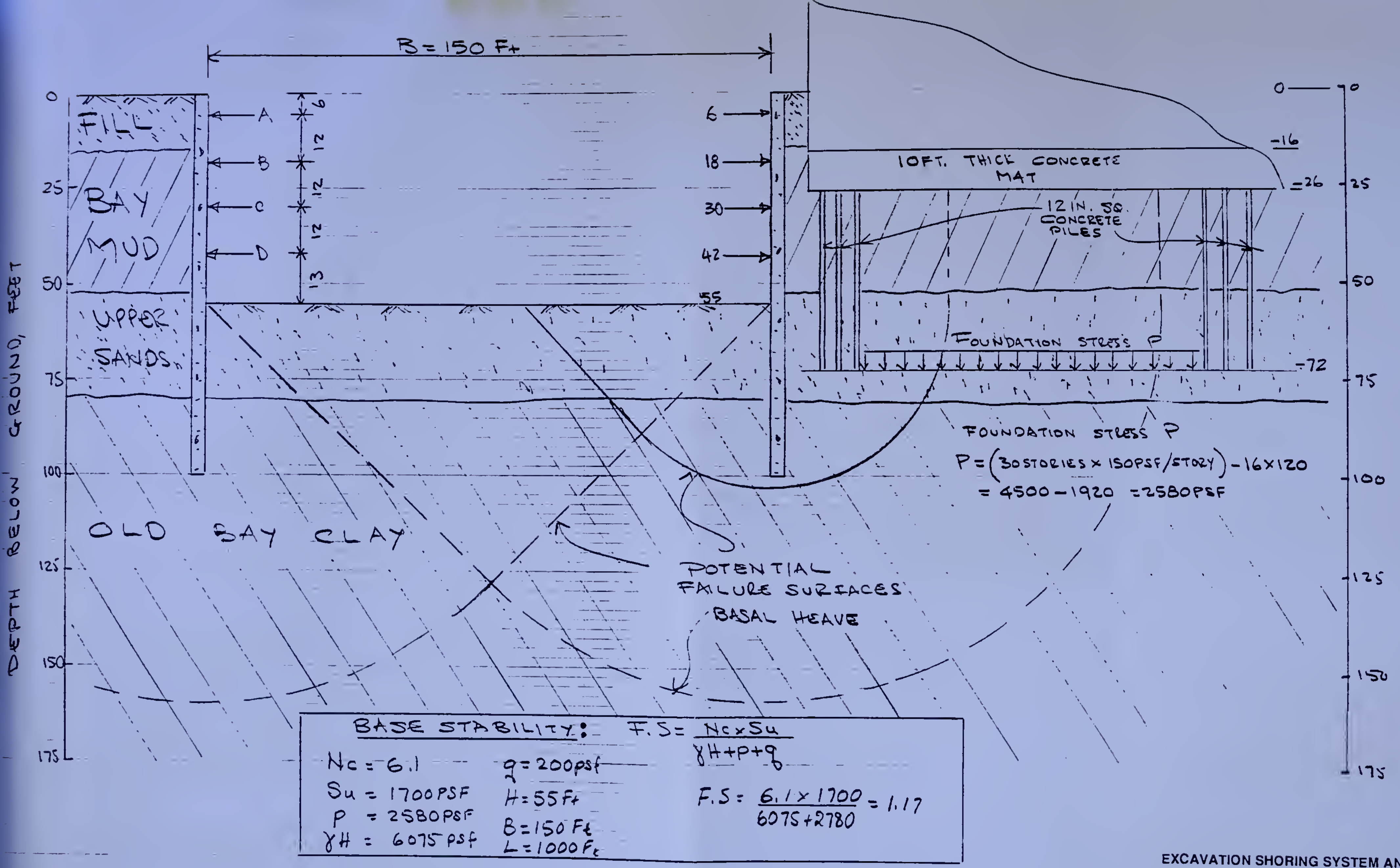
PLATE 6-8



FOUNDATION STRESS P

P = (30 STORIES × 150PSF/STORY) − 16×120

= 4500 − 1920 = 2580PSF

BASE STABILITY: $F.S = \frac{N_c \times S_u}{\gamma H + P + q}$

$N_c = 6.1$ $\quad q = 200psf$

$S_u = 1700PSF$ $\quad H = 55Ft$

$P = 2580PSF$ $\quad B = 150Ft$

$\gamma H = 6075psf$ $\quad L = 1000Ft$

$F.S = \frac{6.1 \times 1700}{6075 + 2780} = 1.17$

EXCAVATION SHORING SYSTEM AND BASE STABILITY FOR THE BEALE/MISSION STREET STATION: ALTERNATIVES 2B AND 2C

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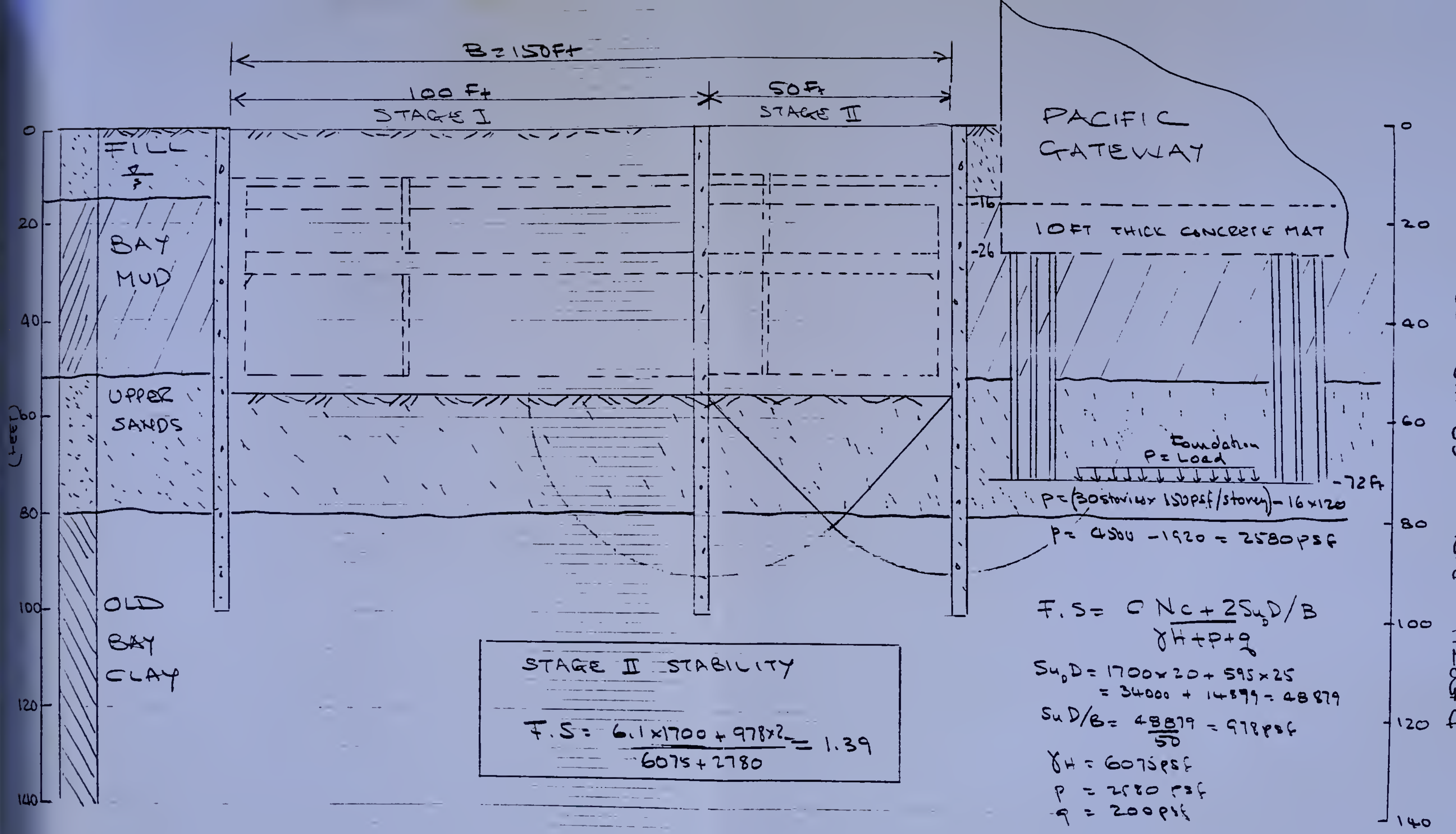
PLATE 6-9



$p = (30\ \text{stories} \times 150\ \text{psf/storey}) - 16 \times 120$

$p = 4500 - 1920 = 2580\ \text{psf}$

$$F.S = \frac{C\,N_c + 2S_{u_D}D/B}{\gamma H + p + q}$$

$S_{u_D}D = 1700 \times 20 + 595 \times 25$

$= 34000 + 14879 = 48879$

$S_u D/B = \frac{48879}{50} = 978\ \text{psf}$

$\gamma H = 6075\ \text{psf}$

$p = 2580\ \text{psf}$

$q = 200\ \text{psf}$

STAGE II STABILITY

$$F.S = \frac{6.1 \times 1700 + 978 \times 2}{6075 + 2780} = 1.39$$

STAGED EXCAVATION TO IMPROVE EXCAVATION STABILITY: ALTERNATIVES 2B AND 2C

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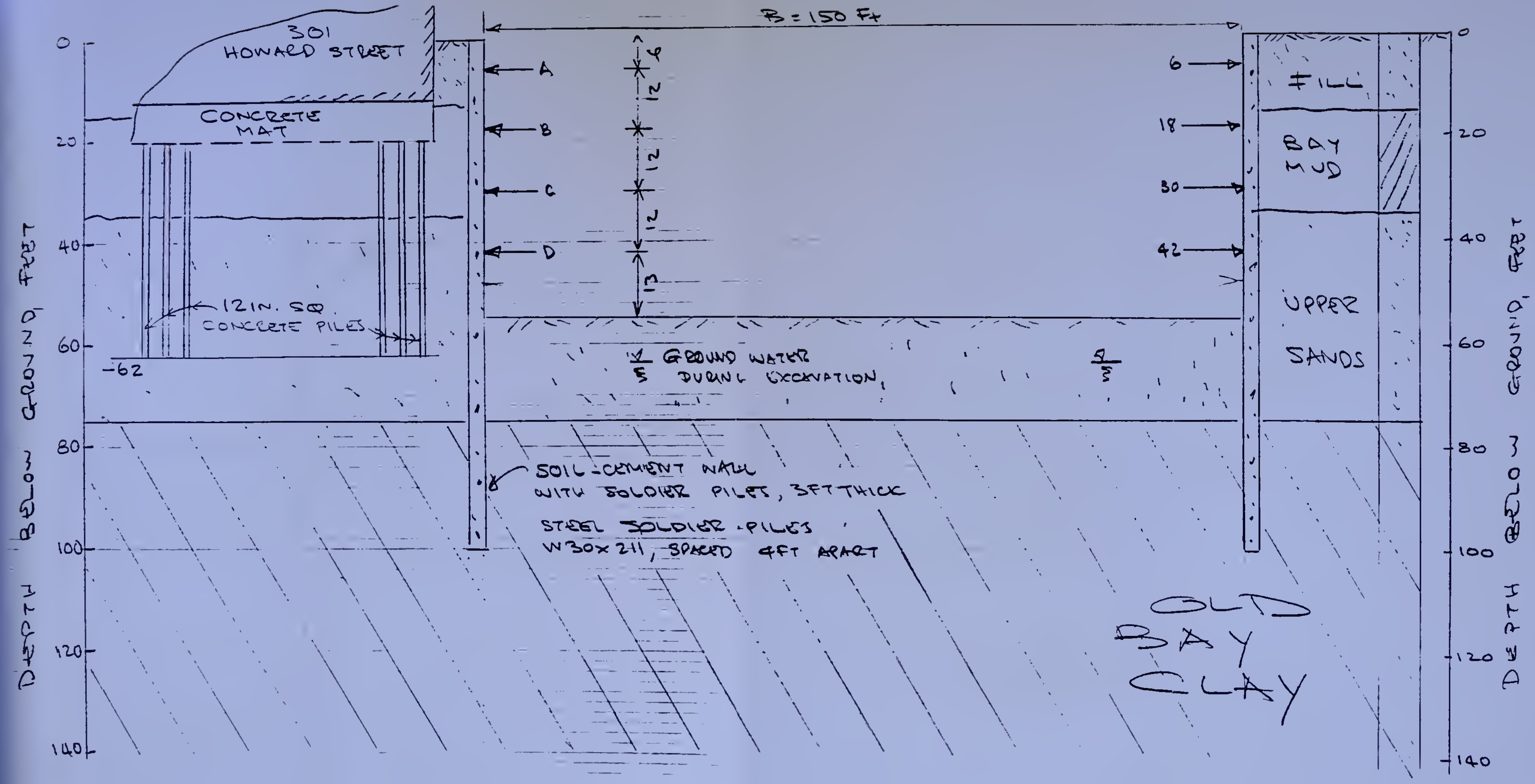
PLATE 6-10



EXCAVATION AND SHORING SYSTEM, CALTRAIN TERMINAL, ALTERNATIVE 2C EXCAVATIONS NEAR HOWARD STREET

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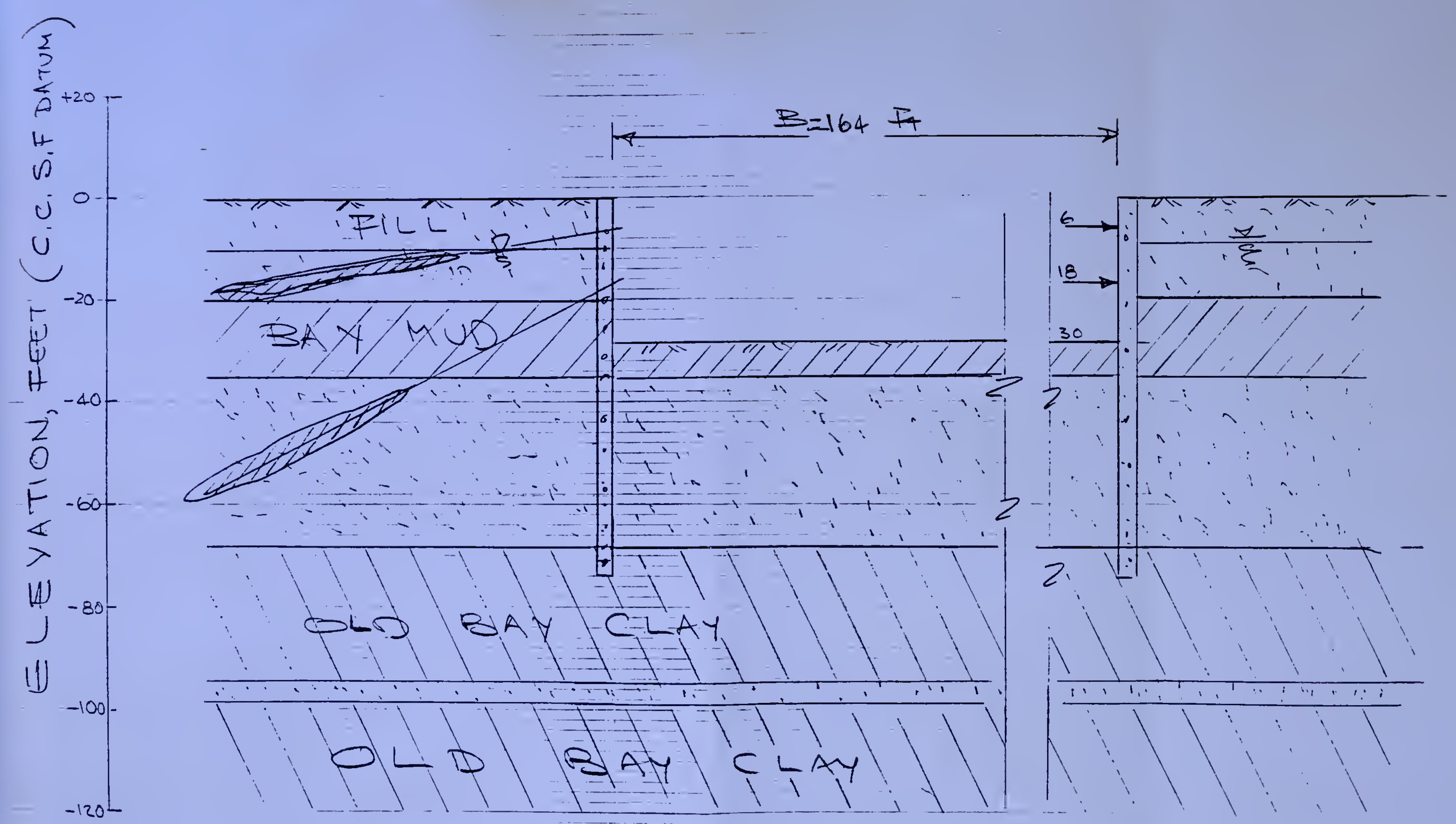
PLATE 6-11



EXCAVATION AND SHORING SYSTEM:
TRANSBAY TERMINAL: ALTERNATIVE 3B

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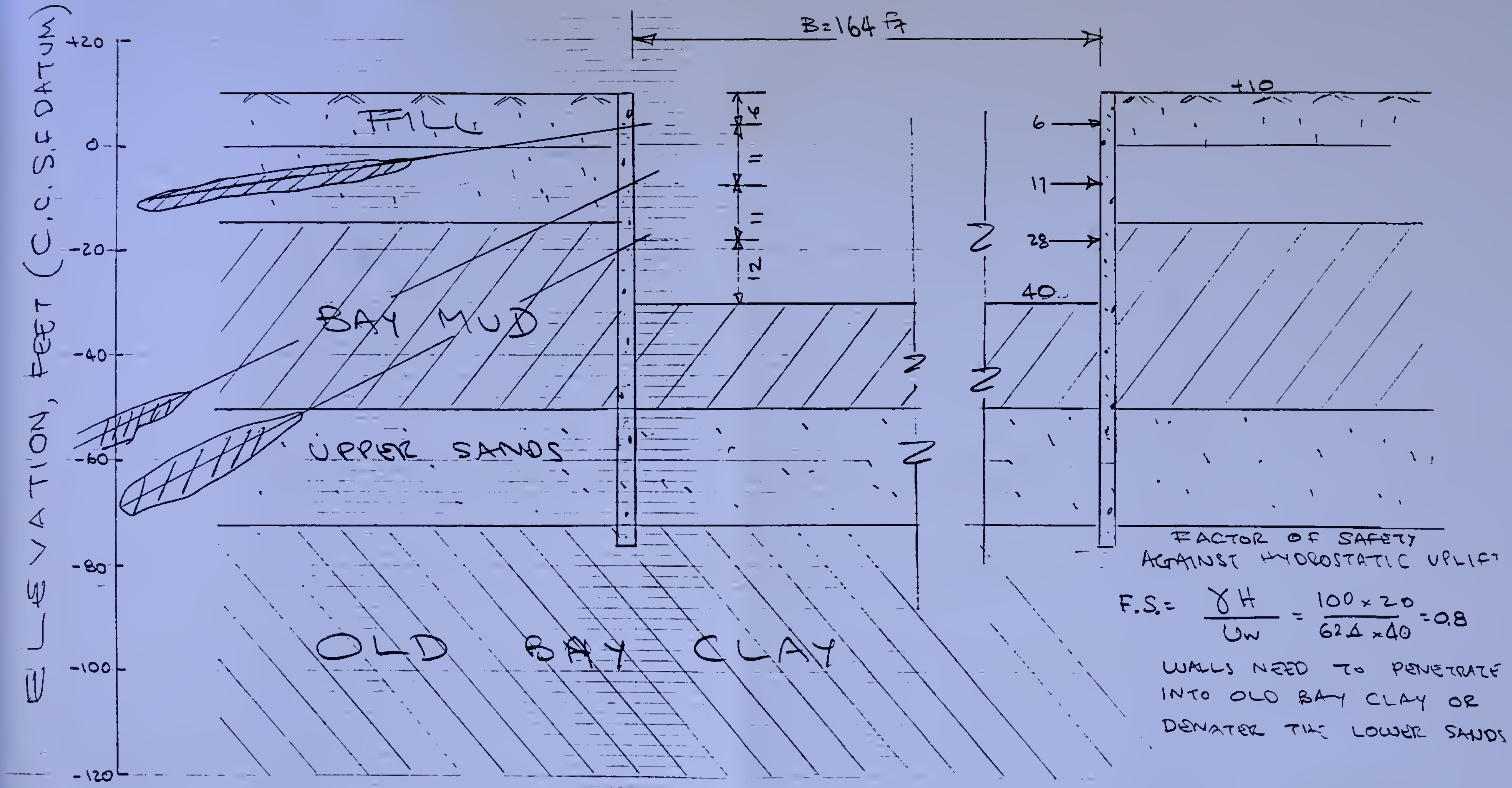
PLATE 6-12



FACTOR OF SAFETY AGAINST HYDROSTATIC UPLIFT

$$F.S. = \frac{\gamma H}{U_w} = \frac{100 \times 20}{62.4 \times 40} = 0.8$$

WALLS NEED TO PENETRATE INTO OLD BAY CLAY OR DEWATER THE LOWER SANDS

EXCAVATION AND SHORING SYSTEM:
TRANSBAY TERMINAL: ALTERNATIVE 3B

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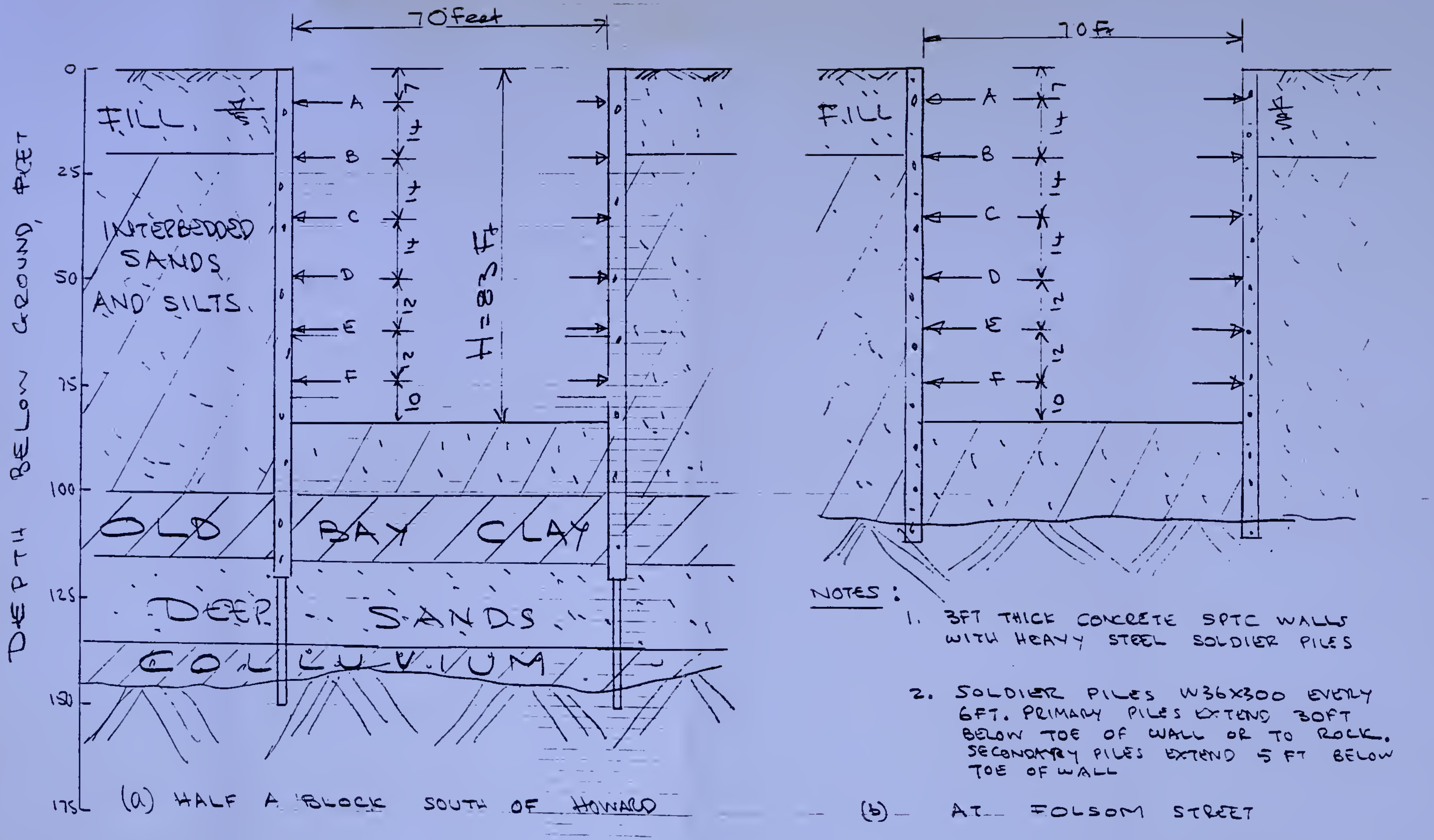
PLATE 6-13



(a) HALF A BLOCK SOUTH OF HOWARD

(b) AT FOLSOM STREET

NOTES:

1. 3FT THICK CONCRETE SPTC WALLS WITH HEAVY STEEL SOLDIER PILES

2. SOLDIER PILES W36X300 EVERY 6FT. PRIMARY PILES EXTEND 30FT BELOW TOE OF WALL OR TO ROCK. SECONDARY PILES EXTEND 5 FT BELOW TOE OF WALL

EXCAVATION AND BRACING SYSTEM FOR EXCAVATIONS BETWEEN HOWARD AND FOLSOM STREETS: ALTERNATIVE 2A-C&C

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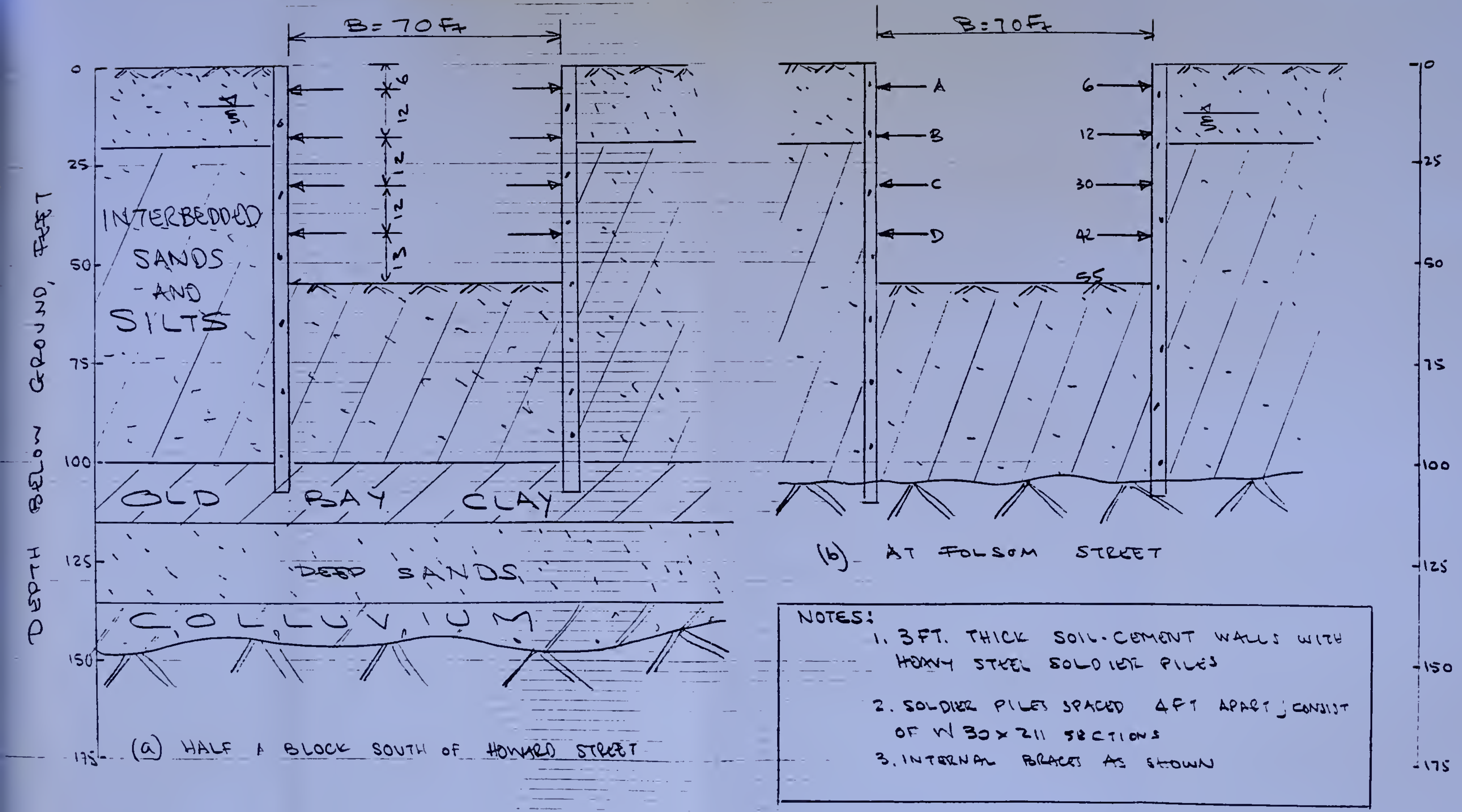
PLATE 6-14



(a) HALF A BLOCK SOUTH OF HOWARD STREET

(b) AT FOLSOM STREET

NOTES:

1. 3 FT. THICK SOIL-CEMENT WALLS WITH HEAVY STEEL SOLDIER PILES
2. SOLDIER PILES SPACED 4 FT APART, CONSIST OF W 30 x 211 SECTIONS
3. INTERNAL BRACES AS SHOWN

**EXCAVATION AND BRACING SYSTEM:
ALTERNATIVES 2B-C&C AND 2C-C&C
EXCAVATIONS NEAR FOLSOM STREET**

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30751-001-003

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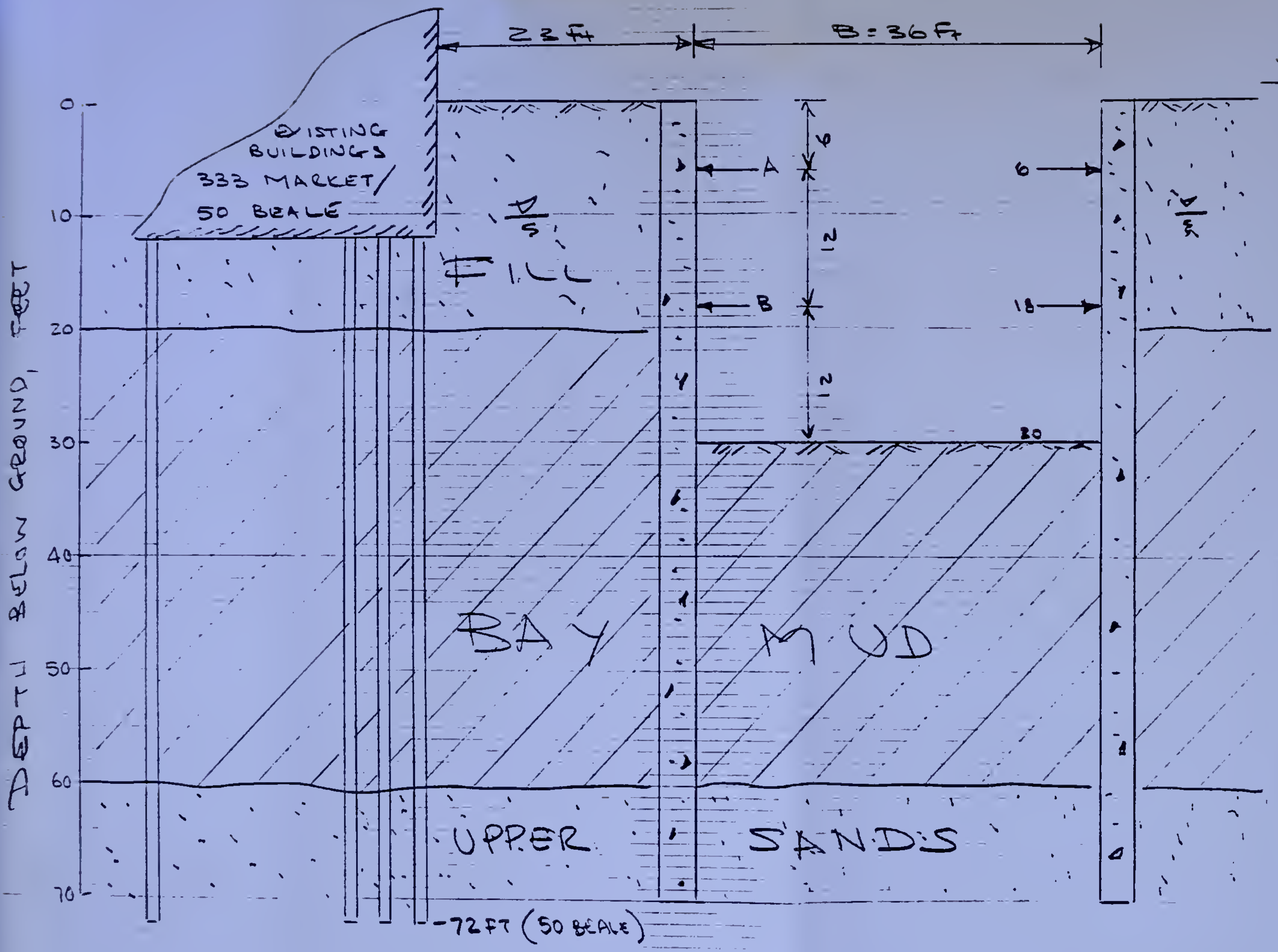
PLATE 6-15



NOTES

1. SHORING SYSTEM CONSISTS OF STEEL SOLDIER PILES WITH IN-SITU SOIL-CEMENT WALLS, 3FT THICK.
2. SOLDIER PILES CONSIST OF W30× 211 SECTIONS SPACED 4 FT APART.
3. WALL IS 70FT DEEP TO ACHIEVE MINIMUM PENETRATION OF 10FT IN UPPER SANDS.

4 ESTIMATED LATERAL DEFORMATIONS 1¼ IN.
(F.S=1.7 S=415)

5. BRACING AS SHOWN.

EXCAVATION AND SHORING SYSTEM:
UNDERGROUND WALKWAY
BETWEEN MISSION AND MARKET STREETS

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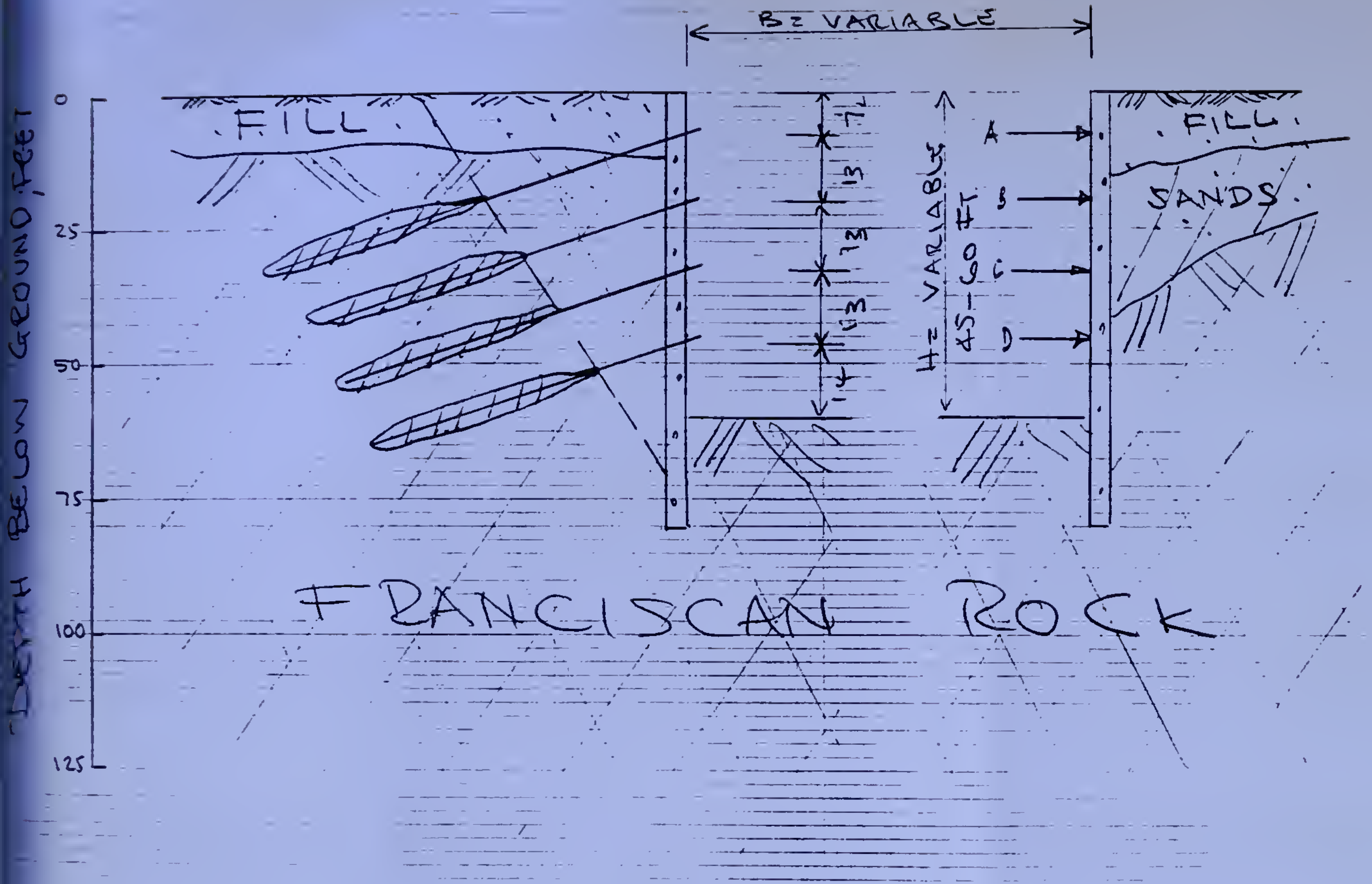
PLATE 6-16



NOTES:

1. SHORING SYSTEM CONSISTS OF SOLDIER PILES WITH WIRE MESH AND SHOTCRETE ON FACE. ROCK ANCHORS OR INTERNAL BRACING AS SHOWN
2. WALERS AT SUPPORT LEVELS TO TO ACT WITH SOLDIER PILES TO FORM A LATTICE SYSTEM TO CONTROL POTENTIAL ROCKFALLS
3. SOLDIER PILES TO PENETRATE UP TO 20FT INTO ROCK, DEPENDING ON ROCK QUALITY
4. FOR CONTROL OF GROUNDWATER SOME GROUTING MAY BE REQUIRED BEHIND THE FACE OF THE EXCAVATION
5. SOLDIER PILES SPACED 6FT ON CENTERS, W24x131
6. SPECIAL CONDITIONS UNDER HARRISON BRIDGE AND IN FRONT OF BAY BRIDGE ANCHORAGE

EXCAVATION AND SHORING SYSTEM IN THE VICINITY OF HARRISON STREET: ALTERNATIVE 2A-C&C

December 1995
30751-001-003

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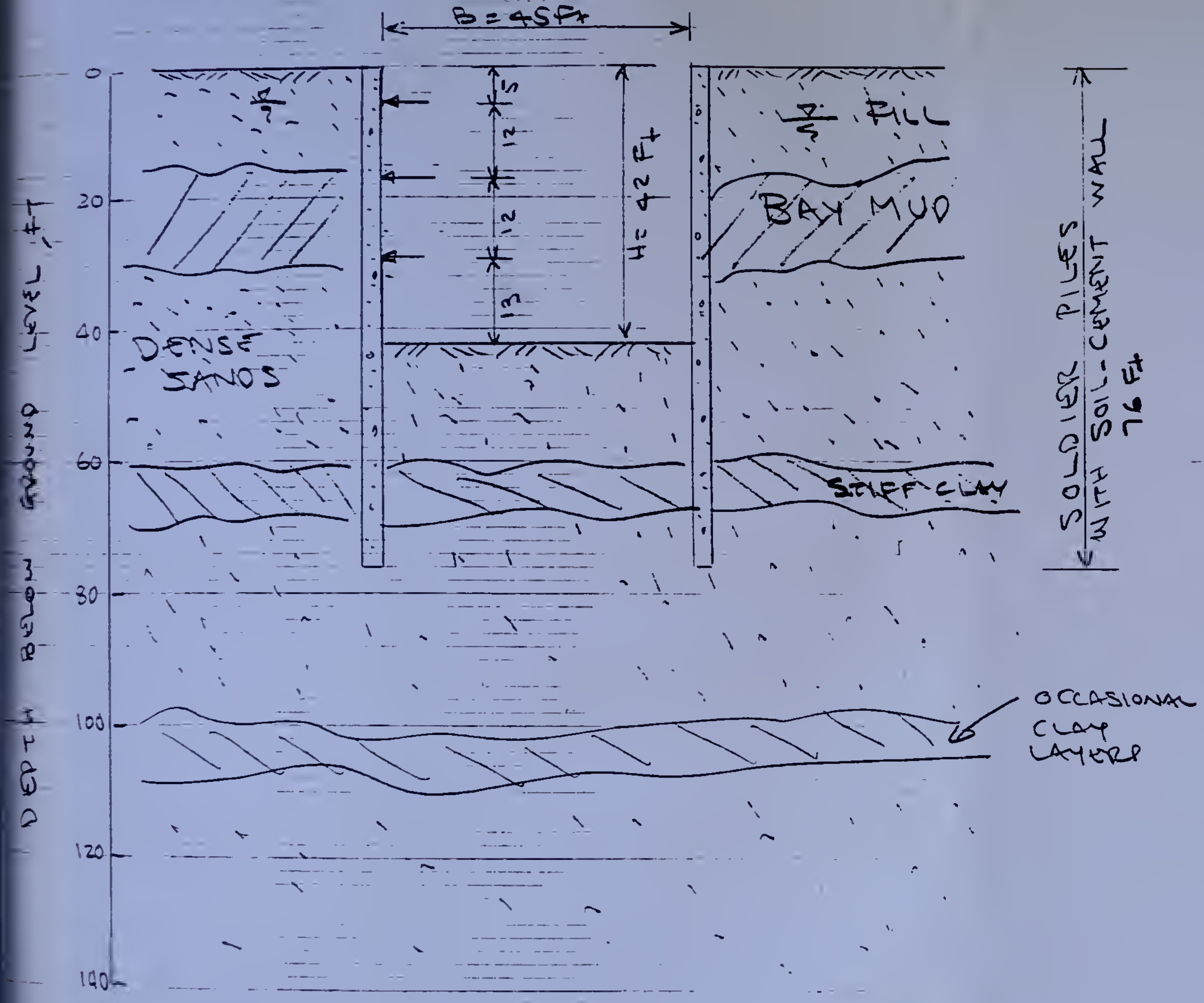
Caltrain S.F. Downtown Station
Relocation EIS/EIR Project

DAMES & MOORE

PLATE 6-17

TYPICAL GROUND CONDITIONS
BRYANT TO BRANNAN



NOTES

1. SHORING SYSTEM CONSISTS OF STEEL SOLDIER PILES AND SOIL-CEMENT WALL
2. WALL IS 3 FT THICK SOIL-CEMENT
4. SOLDIER PILES ARE WF SECTIONS SPACED 4FT ON CENTER, WF 24×94
5. THREE LEVELS OF STRUTS MINIMUM AS SHOWN
6. DEPTH OF THE WALL ASSUMED 80FT DEEP TO TIE INTO AN IMPERMEABLE LAYER BELOW EXCAVATION LEVEL TO CUT-OFF SEEPAGE INTO EXCAVATION
7. DEPTH TO ROCK VARIES FROM 60FT NEAR BRYANT TO 180FT AT BRANNAN AND EMBARCADERO

EXCAVATION AND SHORING SYSTEM - SOUTH BEACH AREA: ALTERNATIVE 2A-C&C

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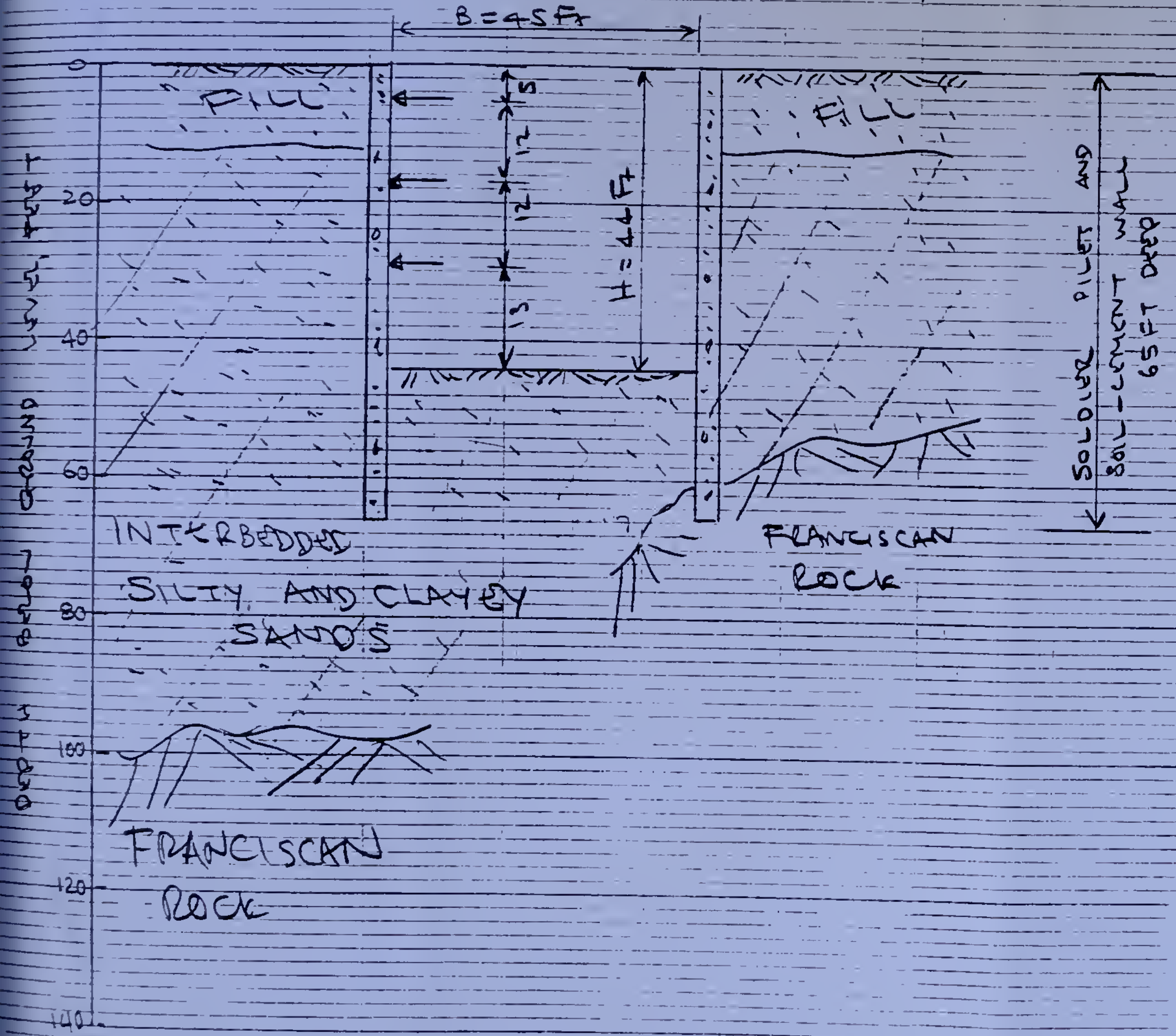
PLATE 6-18

GROUND CONDITIONS ALONG TOWNSEND
NEAR FIRST STREET



B = 45 Ft
FILL
FILL
5
12
12
15
H = 44 Ft
SOLDIER PILES AND SOIL-CEMENT WALL 65 FT DEEP
INTERBEDDED SILTY AND CLAYEY SANDS
FRANCISCAN ROCK
FRANCISCAN ROCK
?
0
20
40
60
80
100
120
140
DEPTH BELOW GROUND SURFACE (FEET)

NOTES:

1. SHORING SYSTEM CONSISTS OF STEEL SOLDIER PILES AND SOIL-CEMENT WALL
2. WALL IS 3FT THICK SOIL-CEMENT, MIXED IN-SITU
3. SOLDIER PILES ARE SPACED 4Ft ON CENTER. CONSIST OF WF 24 x 94
4. THREE LEVELS OF STRUTS MINIMUM AS SHOWN
5. DEPTH OF THE WALL ASSUMED TO BE 65 FT OR UNTIL IT PENETRATES INTO A REASONABLY IMPERMEABLE LAYER
7. DEPTH TO ROCK VARIES FROM 60 FT TO 100 FT

EXCAVATION AND SHORING SYSTEM ALONG TOWNSEND STREET: AREAS BETWEEN FIRST AND COLIN P. KELLY JR. STREETS AND BETWEEN THIRD AND FOURTH STREETS: ALTERNATIVE 2T

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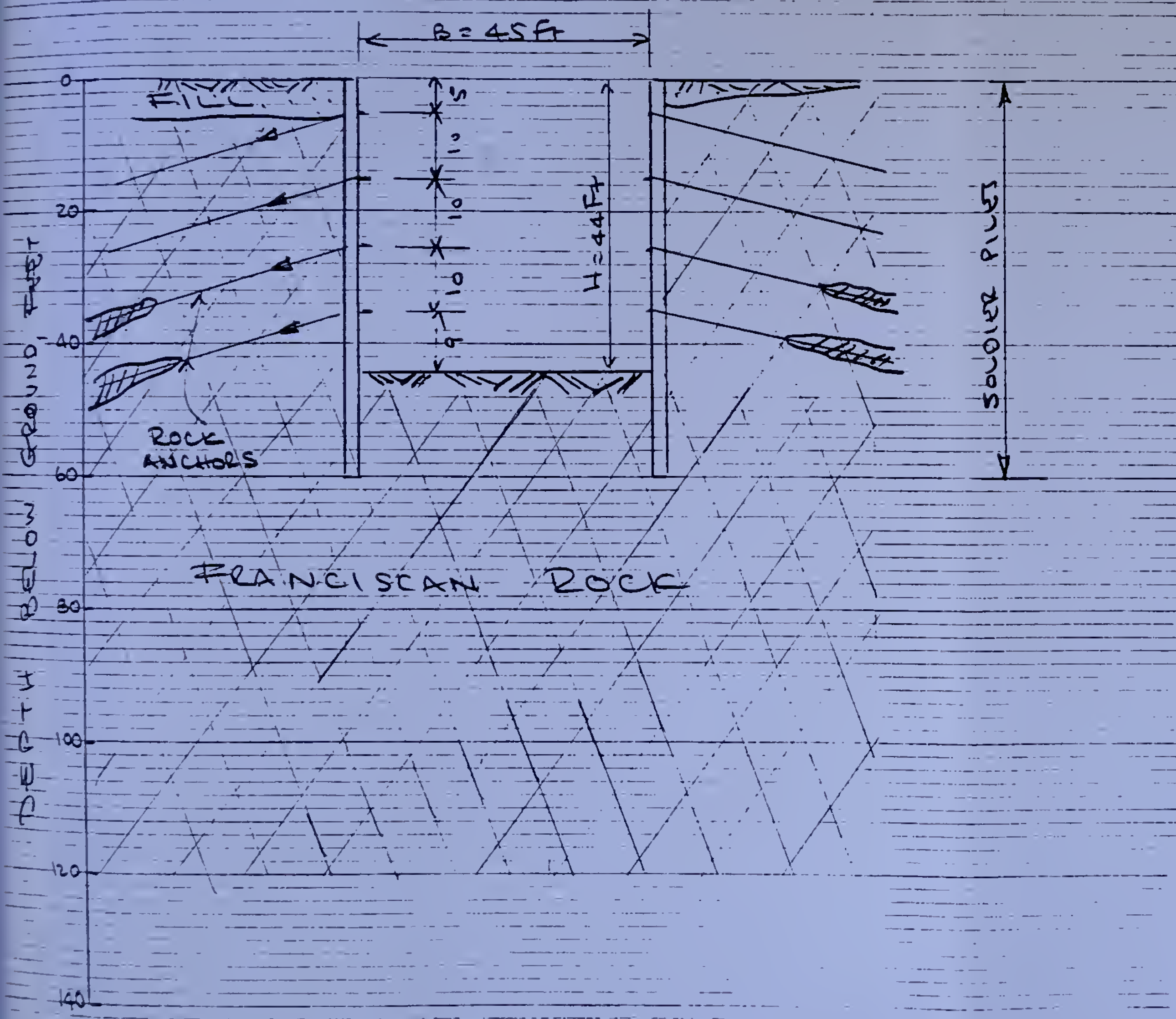
PLATE 6-19

GROUND CONDITIONS ALONG TOWNSEND
BETWEEN SECOND AND THIRD STREETS



NOTES:

1. SHORING CONSISTS OF STEEL SOLDIER PILES AND WALES WITH WIRE MESH AND SHOTCRETE ON FACE, ROCK ANCHORS FOR LATERAL SUPPORT AT 10 FT VERTICAL AND 10 FT HORIZONTAL SPACINGS
2. SOLDIER PILES TO PENETRATE 15 - 20 FT BELOW SUBGRADE DEPENDING ON PRESENCE OR ABSENCE OF SOFT SOIL-LIKE ZONES BELOW EXCAVATION BASE
3. FOR CONTROL OF GROUTING ASSUME 1 ROW OF GROUT HOLES JUST BEHIND THE SHORING WALL
4. WHERE RIGHT OF WAY CAN NOT BE SECURED FOR ROCK ANCHORS USE INTERNAL BRACING
5. ASSUME SOLDIER PILES AT 6 FT CENTERS W14 x 82

EXCAVATION AND SHORING SYSTEM IN AREA IN THE VICINITY OF SECOND AND THIRD STREETS ALONG TOWNSEND: ALTERNATIVE 2T

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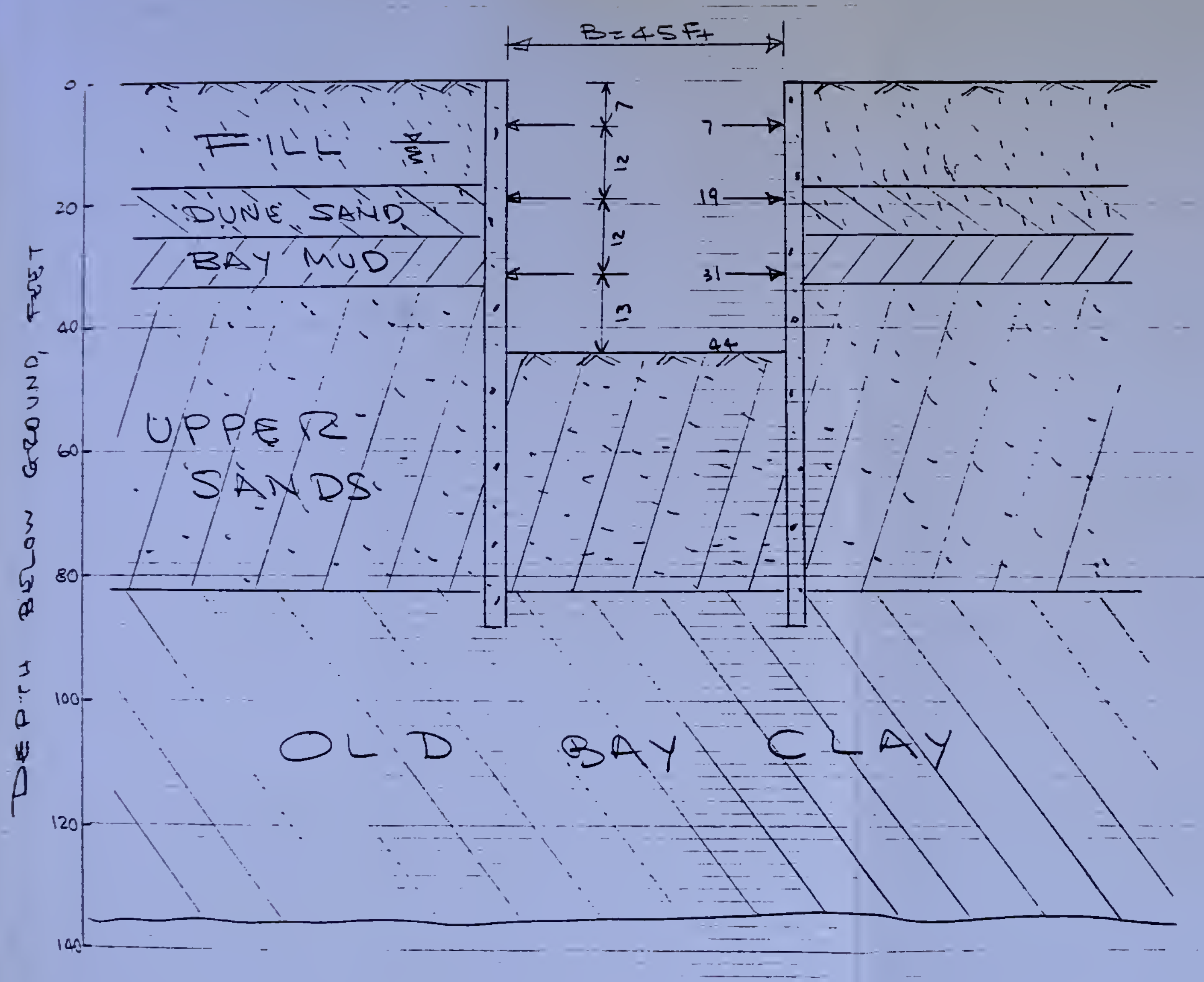
PLATE 6-20



NOTES:

1. SHORING CONSISTS OF STEEL SOLDIER PILES AND SOIL-CEMENT WALLS, 3FT THICK
2. SOLDIER PILES INSTALLED EVERY 4FT.
3. SOLDIER PILES ARE W30x108
4. INTERNAL BRACING AS SHOWN, SPACED 15FT ON CENTER, IN LONGITUDINAL DIRECTION
5. SOIL-CEMENT WALLS PENETRATE 5FT INTO OLD BAY CLAY, ABOUT 90FT DEEP.

EXCAVATION AND SHORING SYSTEM: ALTERNATIVE 3B NEAR HOWARD STREET

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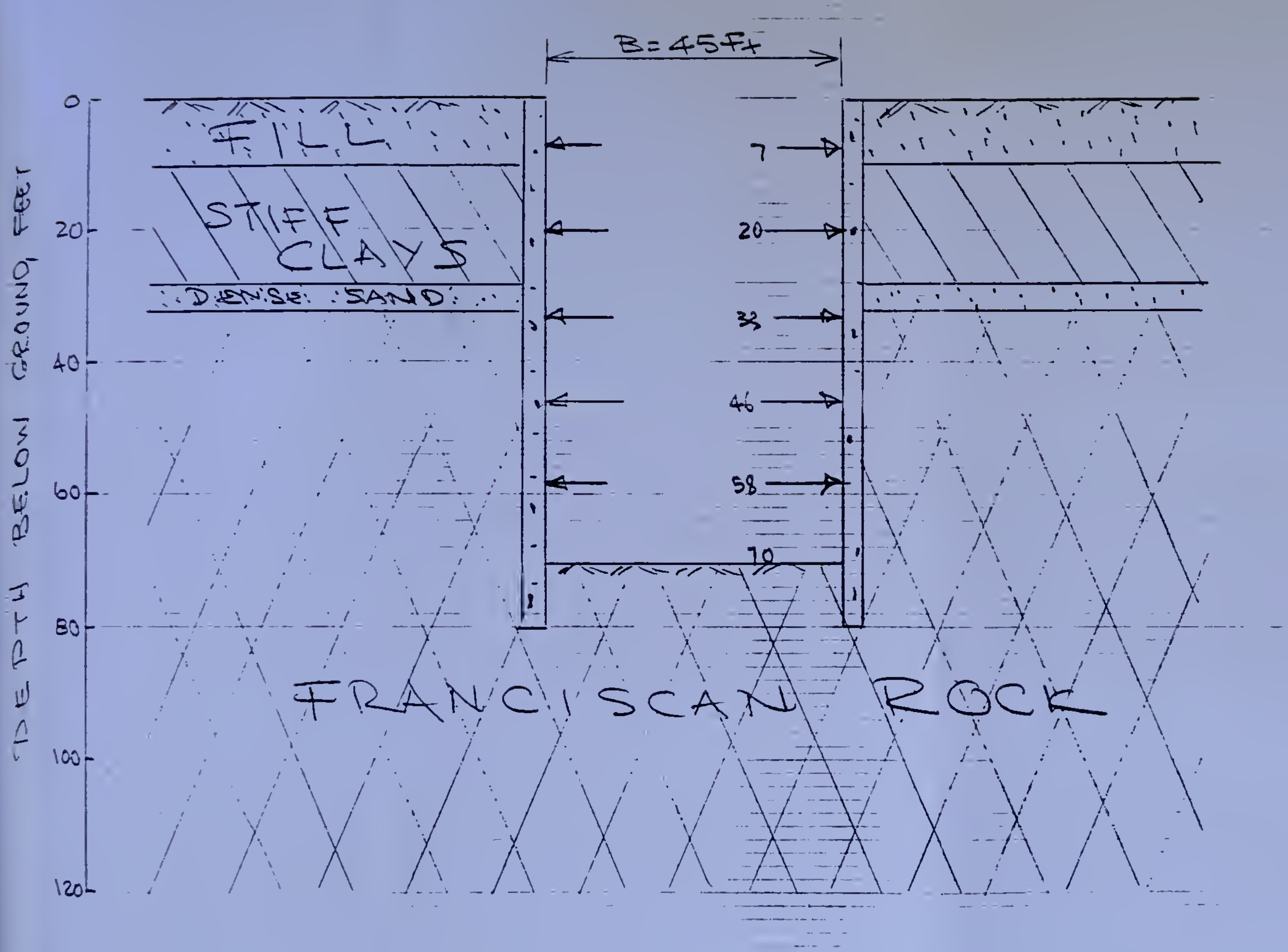

NOTES:

1. SHORING SYSTEM CONSISTS OF CONCRETE CYLINDER PILES TO FORM A TANGENT WALL; OR STEEL SOLDIER PILES INSTALLED IN DRILLED HOLES, WITH LAGGING ABOVE THE ROCK, WIRE MESH AND SHOTCRETE BELOW THE TOP OF ROCK
2. SOLDIER PILES SPACED 5FT ON CENTER. WALL IS 3 FT THICK SOLDIER PILES ARE W30x132
3. FIVE LEVELS OF STRUTS AS SHOWN.
4. SOLDIER PILES TO PENETRATE A MINIMUM OF 5FT IN ROCK WITH RQD>50% AND 15% IN ROCK WITH RQD <50%
5. SOME DEWATERING WILL BE REQUIRED.

EXCAVATION SHORING SYSTEM:
ALTERNATIVES 3A/3B, NEAR BRANNAN STREET

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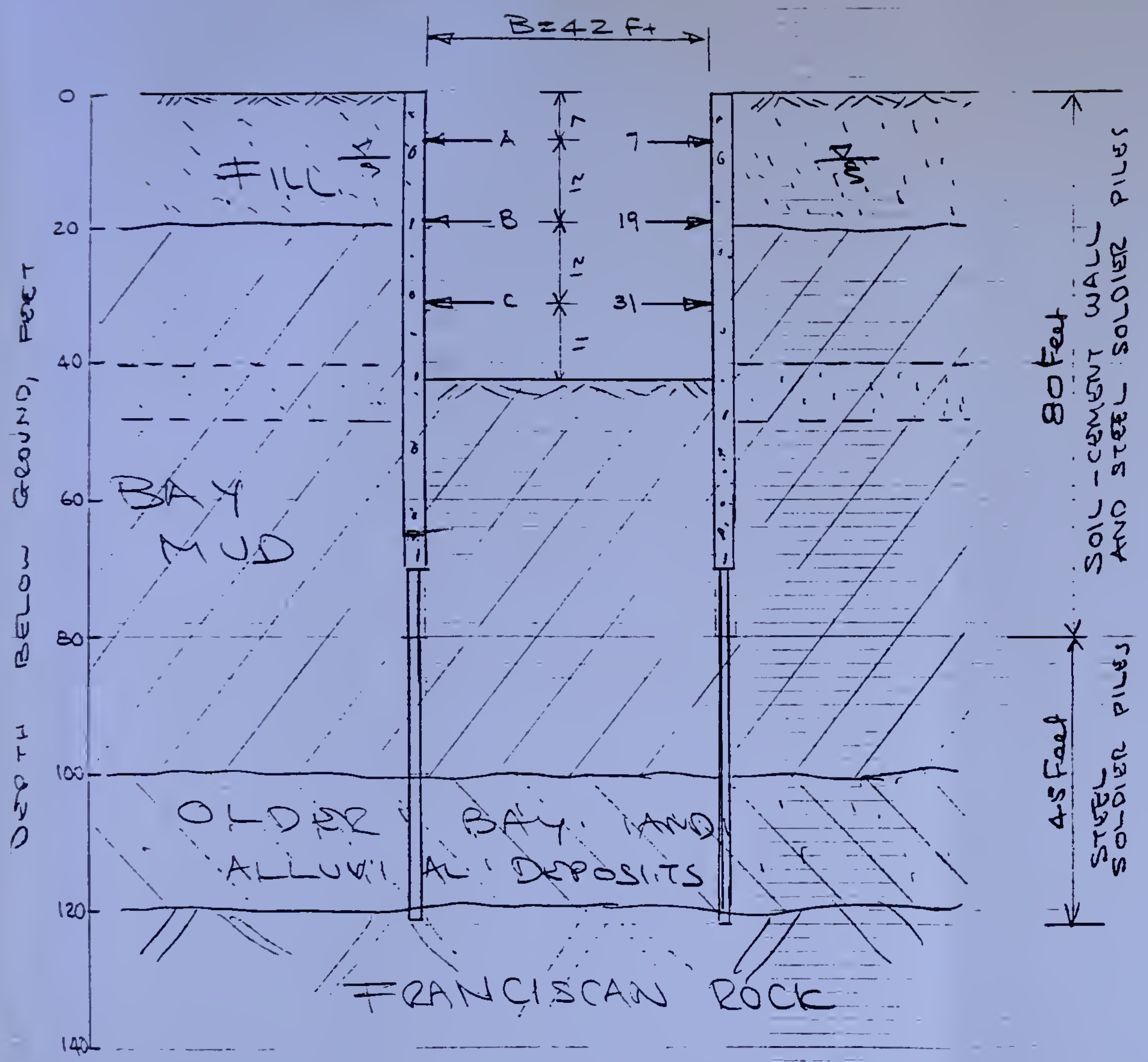
PLATE 6-22



NOTES:

1. SHORING SYSTEM CONSISTS OF STEEL SOLDIER PILES AND SOIL-CEMENT WALL
2. THE SOIL-CEMENT WALL IS 3FT THICK
3. SOLDIER PILES INSTALLED EVERY 4 FT. EVERY OTHER SOLDIER PILE DRIVEN TO COMPETENT STRATA BELOW THE BAY MUD

4 SOLDIER PILES ARE W30x173.

5. INTERNAL BRACING AS SHOWN SPACED 15 FT ON CENTER IN LONGITUDINAL DIRECTION

EXCAVATION SHORING SYSTEM:
ALTERNATIVES 3A/3B-T, BETWEEN FIFTH AND
SIXTH STREETS

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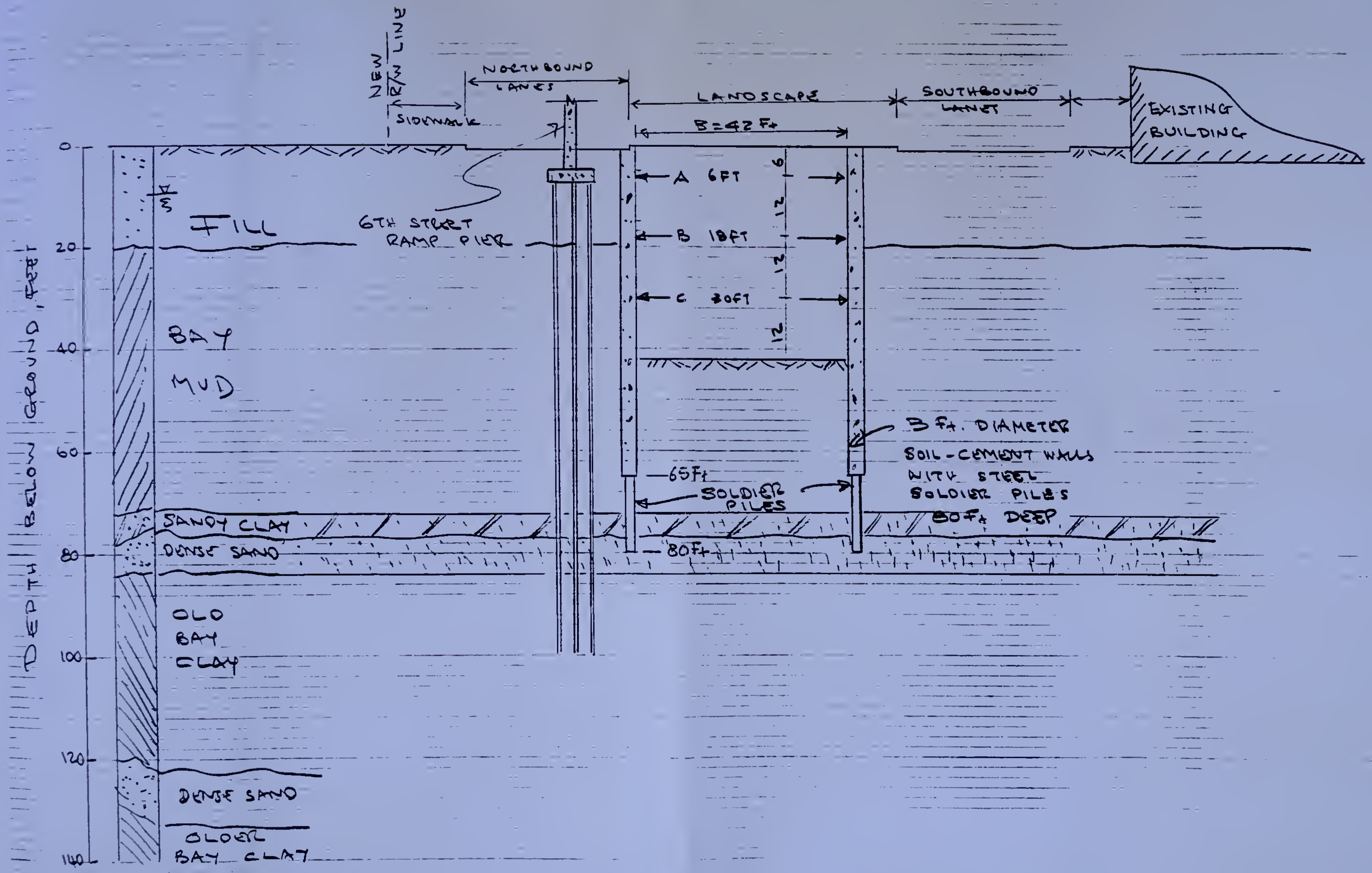
PLATE 6-23



EXCAVATION SHORING SYSTEM.
ALTERNATIVES 3A/3B-T AT SIXTH STREET

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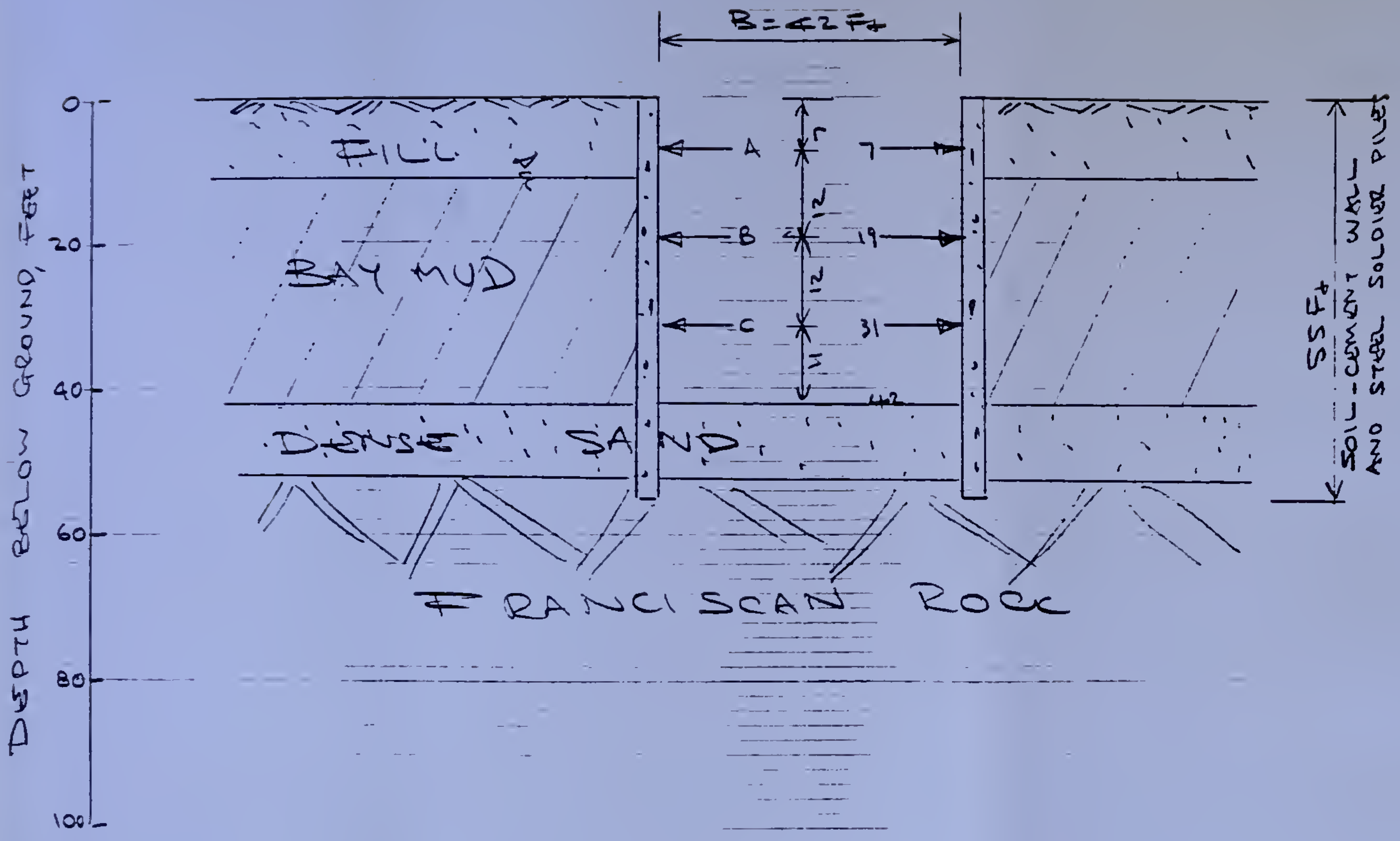
PLATE 6-24



NOTES:

1. SHORING SYSTEM CONSISTS OF STEEL SOLDIER PILES AND SOIL-CEMENT WALL, 3FT THICK
2. SOLDIER PILES SPACED 4FT APART ALONG ALIGNMENT
3. SOLDIER PILES ARE W30×132
4. INTERNAL SPACING AS SHOWN SPACED 15Ft ON CENTERS IN LONGITUDINAL DIRECTION

EXCAVATION SHORING SYSTEM: ALTERNATIVES 3A/3B-T NEAR BERRY STREET

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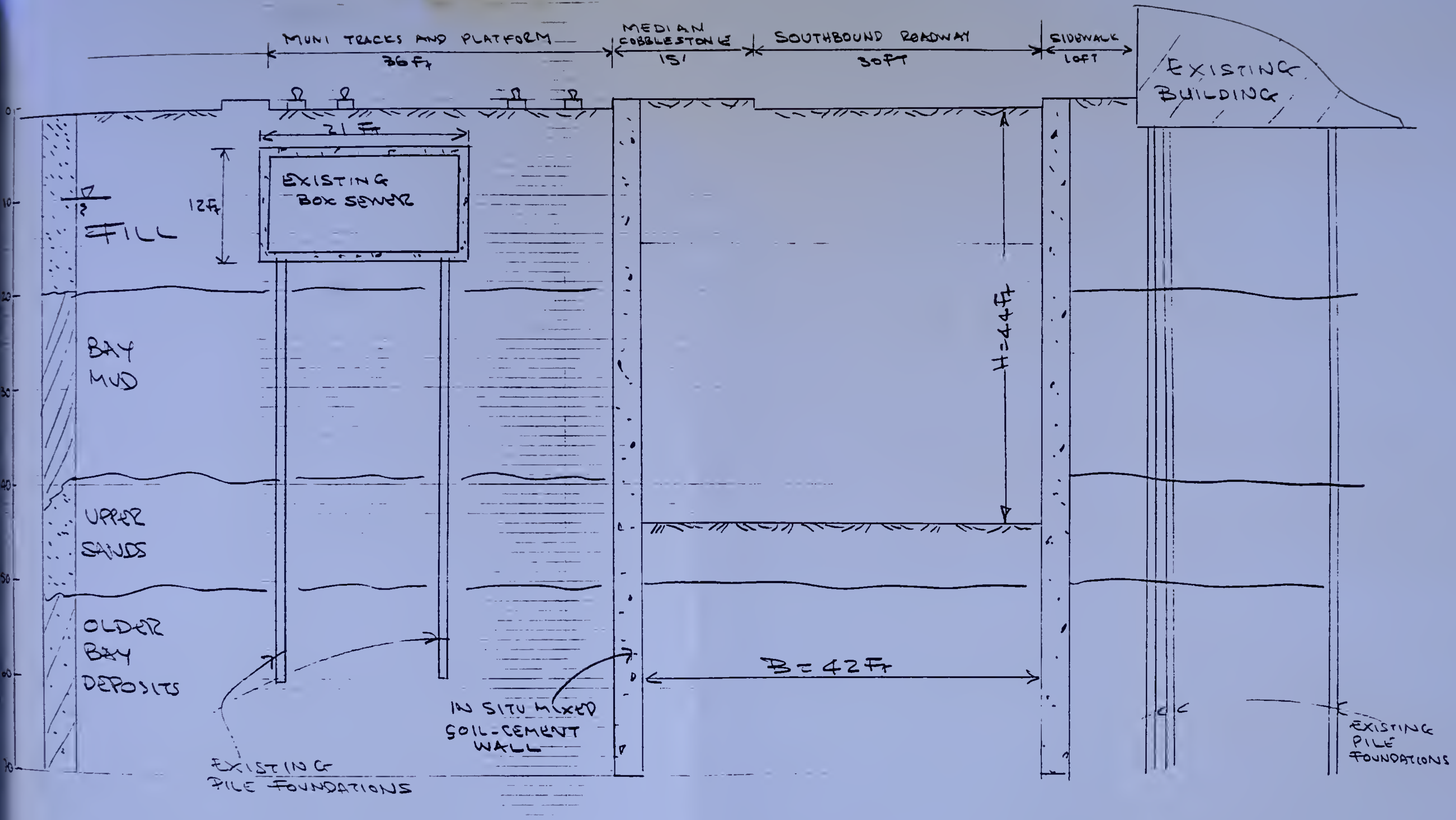
PLATE 6-25



CROSS SECTION SHOWING EXCAVATION AND ADJACENT STRUCTURES ALONG THE EMBARCADERO

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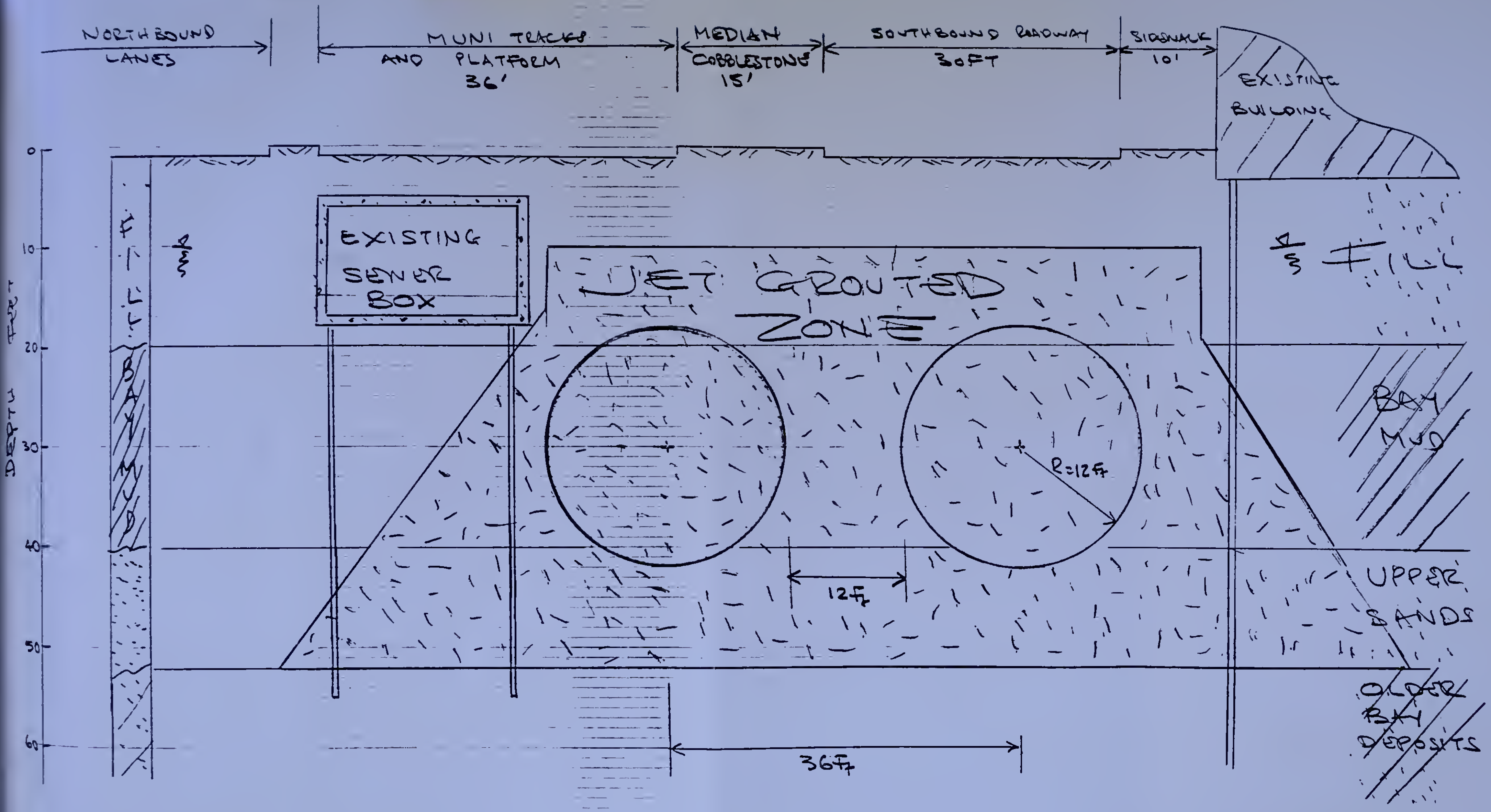
PLATE 6-26



SOFT GROUND TUNNELING ALTERNATIVE ALONG THE EMBARCADERO

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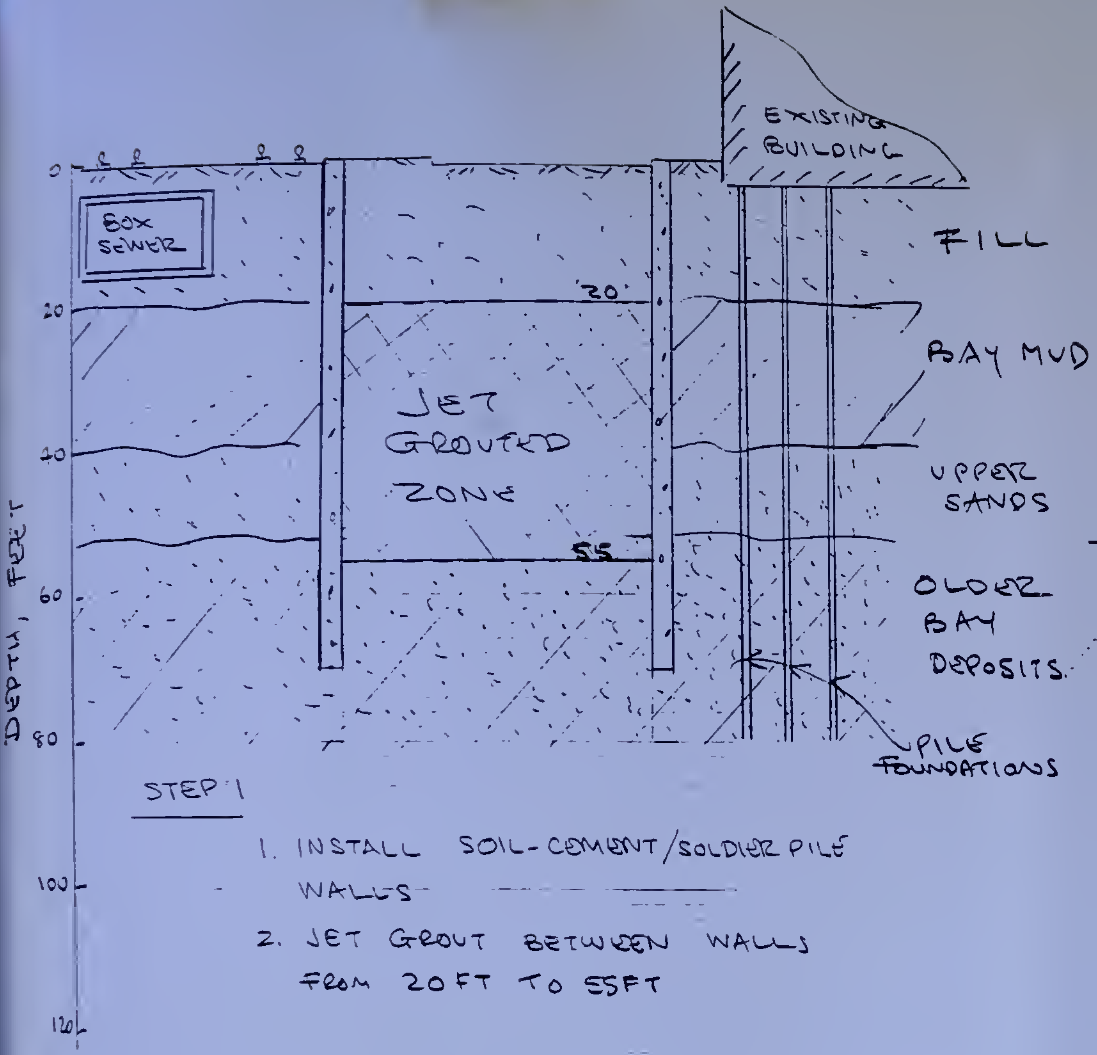
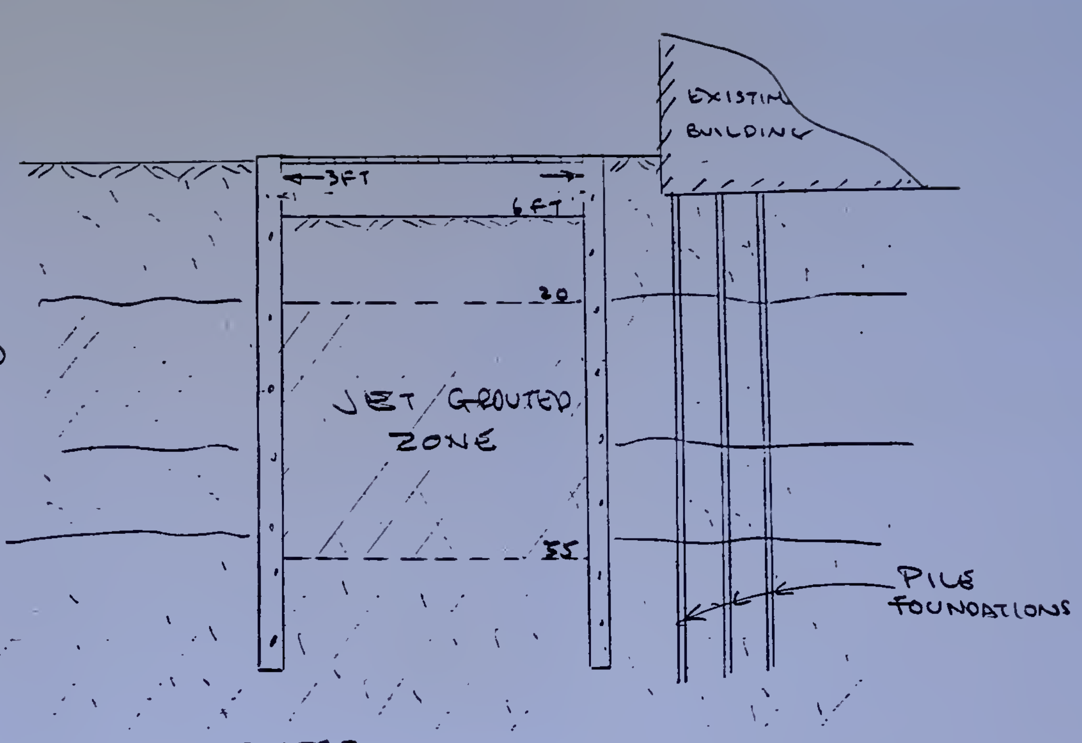
PLATE 6-27



STEP 1

1. INSTALL SOIL-CEMENT/SOLDIER PILE WALLS
2. JET GROUT BETWEEN WALLS FROM 20 FT TO 55 FT



STEP 2

1. EXCAVATE TO A DEPTH OF 6 FT
2. INSTALL FIRST LEVEL OF STRUTS AT 3-FOOT DEPTH; AND PRELOAD THE STRUTS
3. CONSTRUCT STREET DECK
4. OVEREXCAVATE A SHORT SECTION AT EITHER END OF THE ALIGNMENT TO A DEPTH OF 20 FT, TO BE USED AS ACCESS SHAFT FOR EXCAVATING EQUIPMENT.

STAGED EXCAVATION ALTERNATIVE
CUT-AND-COVER ALONG THE EMBARCADERO
(STEPS 1 & 2)

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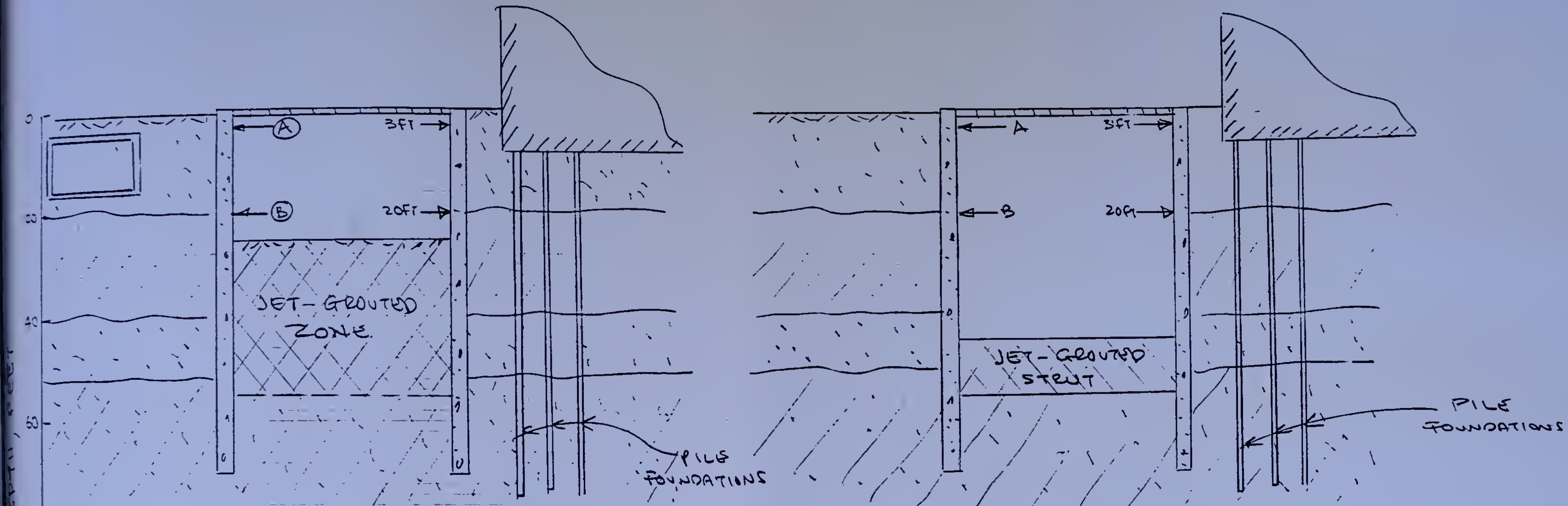
PLATE 6-28a



STEP 3

1. EXCAVATE TO A DEPTH OF 25FT.
2. INSTALL B-LEVEL STRUTS AT DEPTH OF 20 FT

STEP 4

1. EXCAVATE TO GRADE
2. POUR BASE SLAB
3. REMOVE LEVEL B STRUTS
4. CONSTRUCT BOX

STAGED EXCAVATION ALTERNATIVE
CUT-AND-COVER ALONG THE EMBARCADERO
(STEPS 3 & 4)

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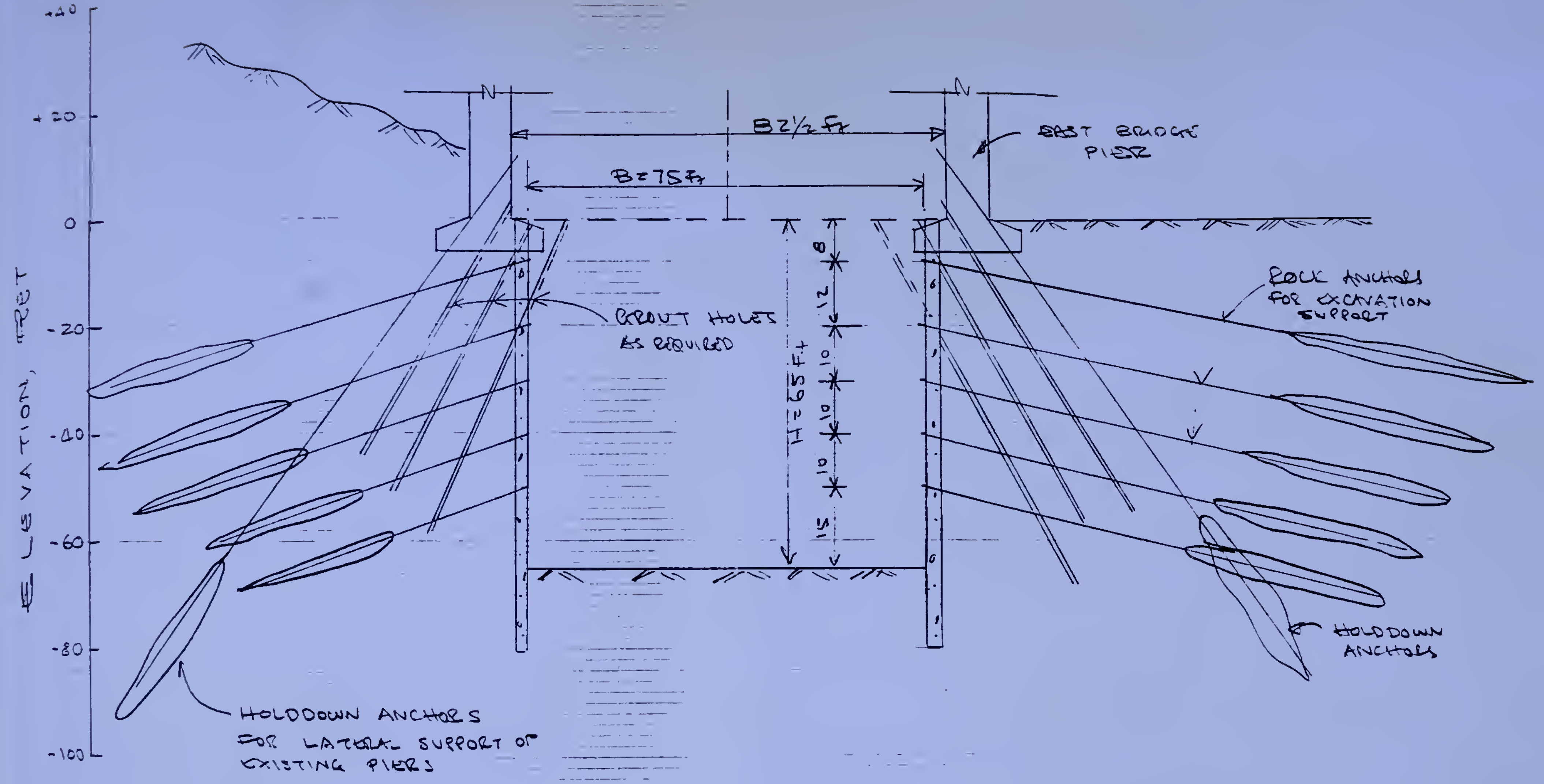
PLATE 6-28b



EXCAVATION UNDER HARRISON STREET BRIDGE: ALTERNATIVE 2A-C&C; DEEP EXCAVATIONS

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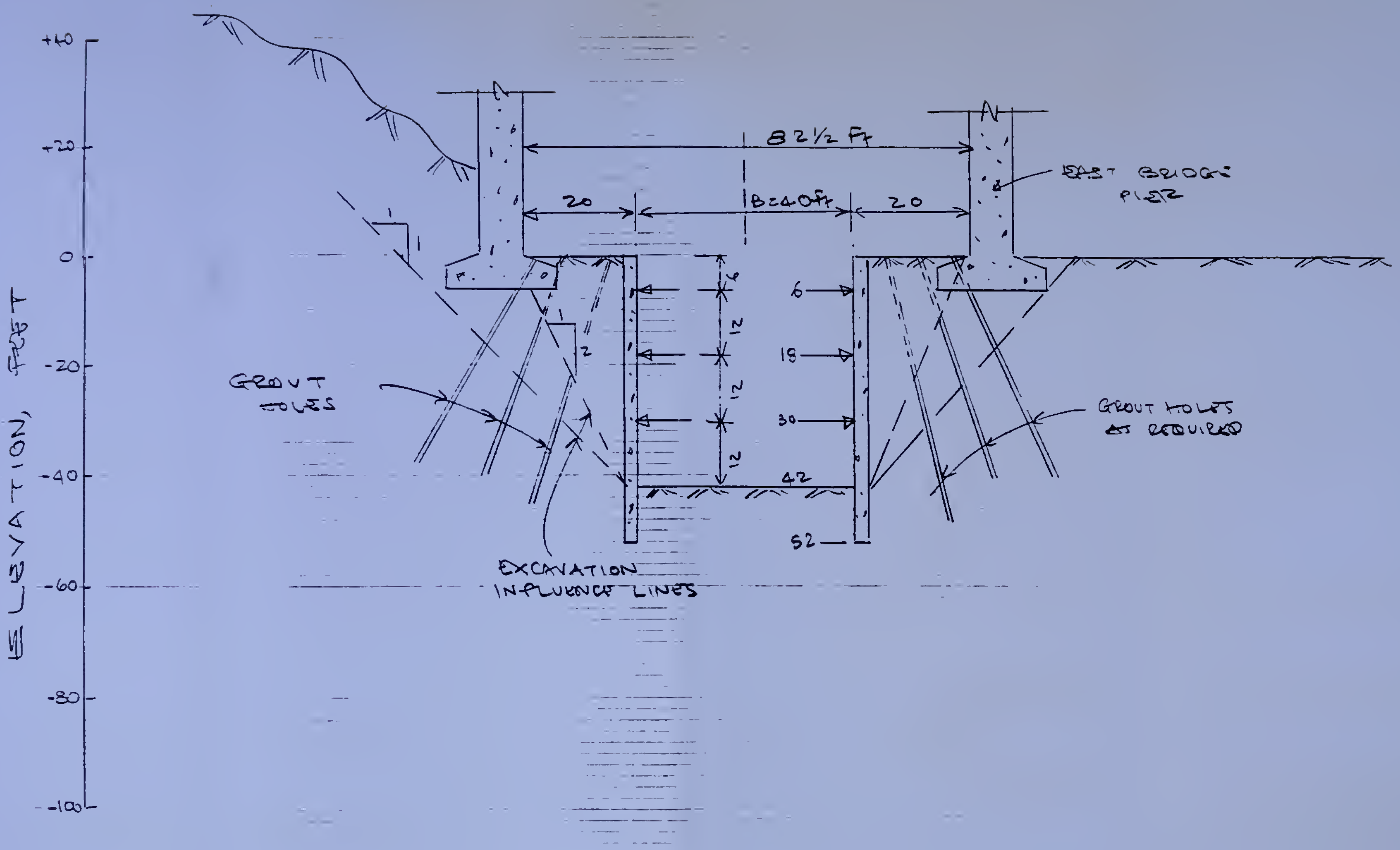
PLATE 6-29



EXCAVATION UNDER HARRISON STREET BRIDGE: ALTERNATIVE 2B-C&C; SHALLOW EXCAVATIONS

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30751-001-003

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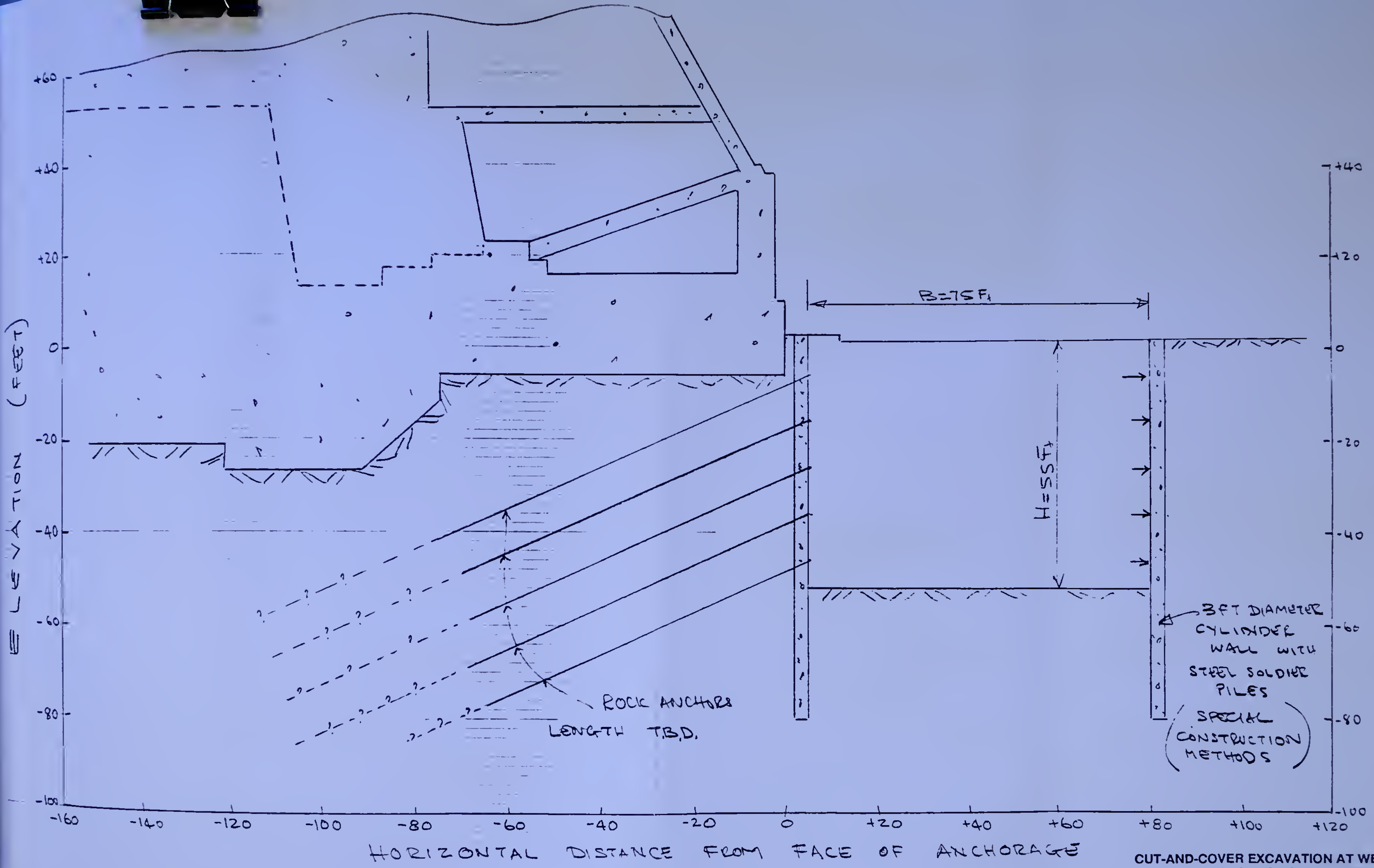
PLATE 6-30



CUT-AND-COVER EXCAVATION AT WEST ANCHORAGE OF BAY BRIDGE: ALTERNATIVES 2A-C&C

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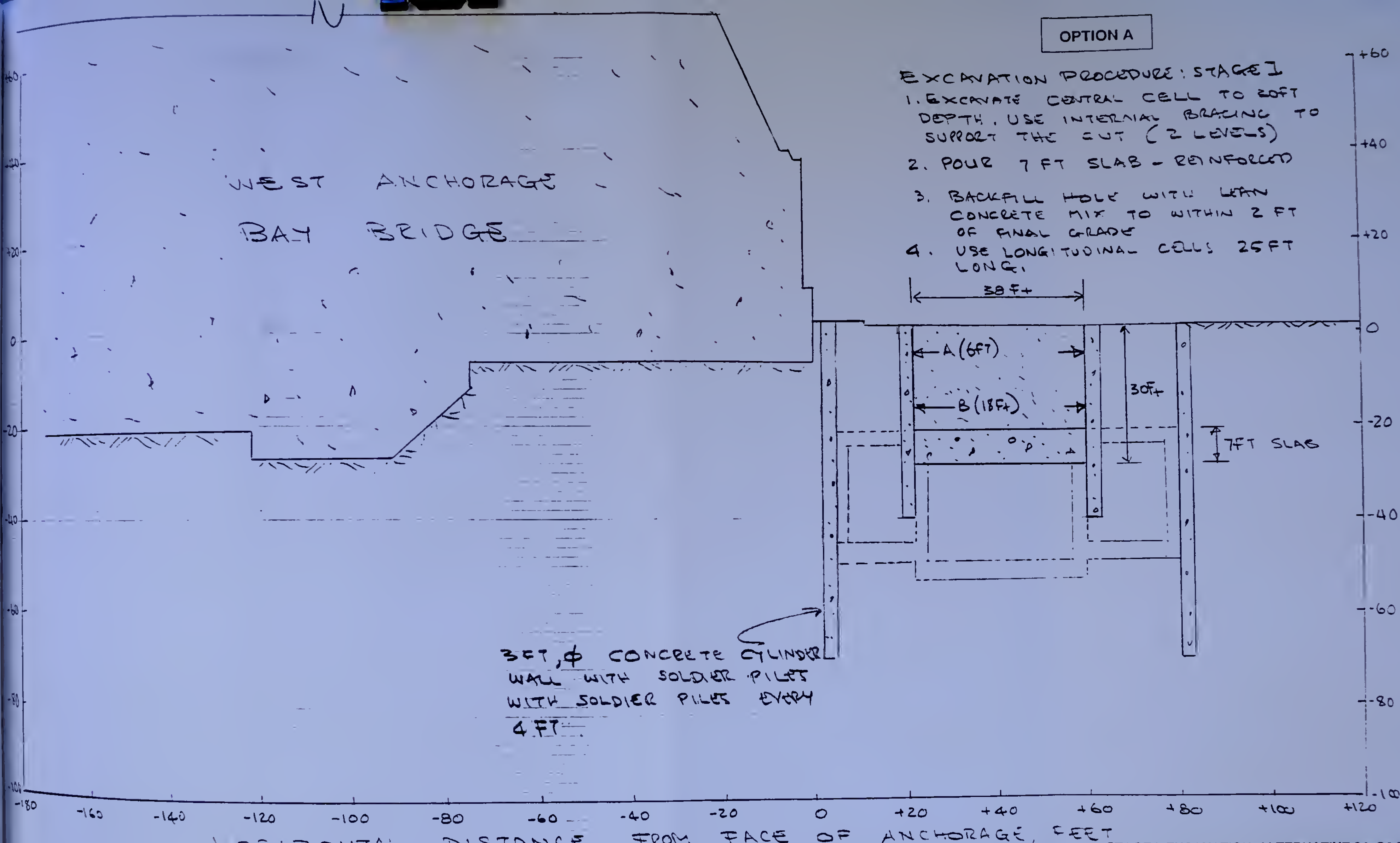
PLATE 6-31



EXCAVATION PROCEDURE: STAGE I

1. EXCAVATE CENTRAL CELL TO 30FT DEPTH. USE INTERNAL BRACING TO SUPPORT THE CUT (2 LEVELS)
2. POUR 7 FT SLAB - REINFORCED
3. BACKFILL HOLE WITH LEAN CONCRETE MIX TO WITHIN 2 FT OF FINAL GRADE
4. USE LONGITUDINAL CELLS 25FT LONG.

STAGE I EXCAVATION, ALTERNATIVE 2A-C&C OPTION A, CUT-AND-COVER CONSTRUCTION: BAY BRIDGE ANCHORAGE AREA

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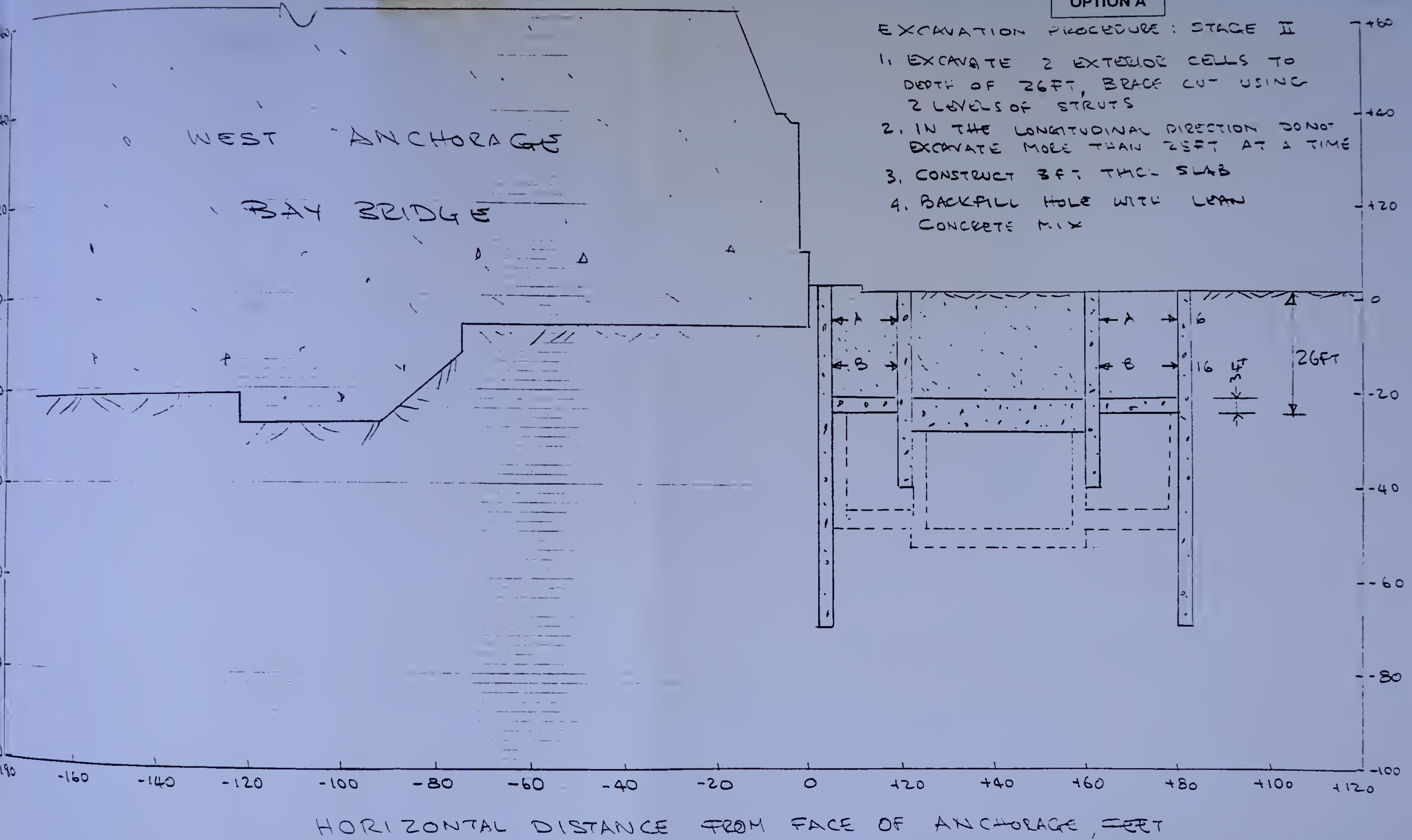
PLATE 6-32a

OPTION A



EXCAVATION PROCEDURE : STAGE II

1. EXCAVATE 2 EXTERIOR CELLS TO DEPTH OF 26FT, BRACE CUT USING 2 LEVELS OF STRUTS
2. IN THE LONGITUDINAL DIRECTION DO NOT EXCAVATE MORE THAN 25FT AT A TIME
3. CONSTRUCT 3FT THICK SLAB
4. BACKFILL HOLE WITH LEAN CONCRETE MIX

STAGE II EXCAVATION, ALTERNATIVE 2A-C&C
OPTION A, CUT AND COVER CONSTRUCTION:
BAY BRIDGE ANCHORAGE AREA

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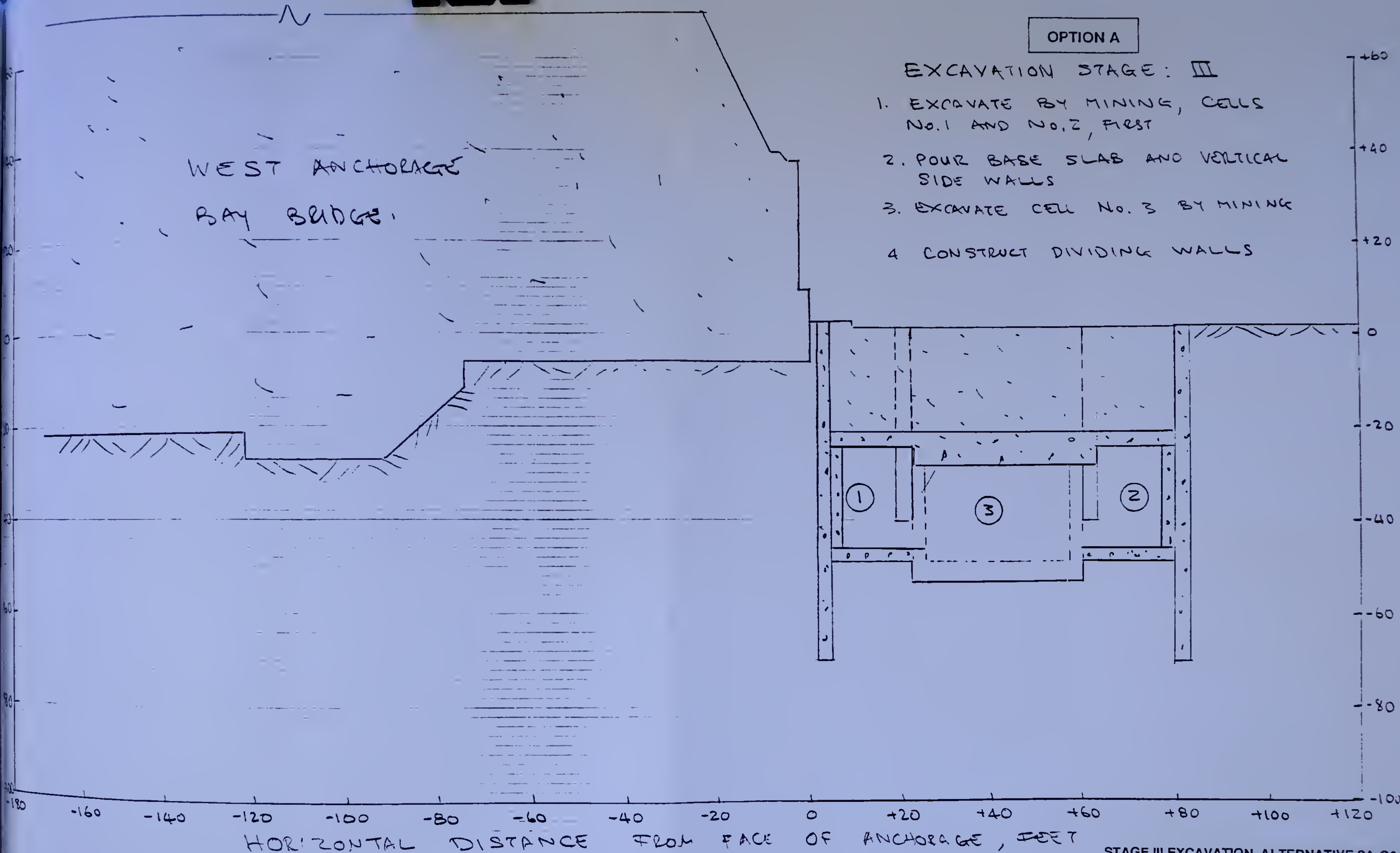
PLATE 6-32b

OPTION A

EXCAVATION STAGE: III

1. EXCAVATE BY MINING, CELLS No. 1 AND No. 2, FIRST
2. POUR BASE SLAB AND VERTICAL SIDE WALLS
3. EXCAVATE CELL No. 3 BY MINING
4 CONSTRUCT DIVIDING WALLS



STAGE III EXCAVATION, ALTERNATIVE 2A-C&C OPTION A, CUT AND COVER CONSTRUCTION: BAY BRIDGE ANCHORAGE AREA

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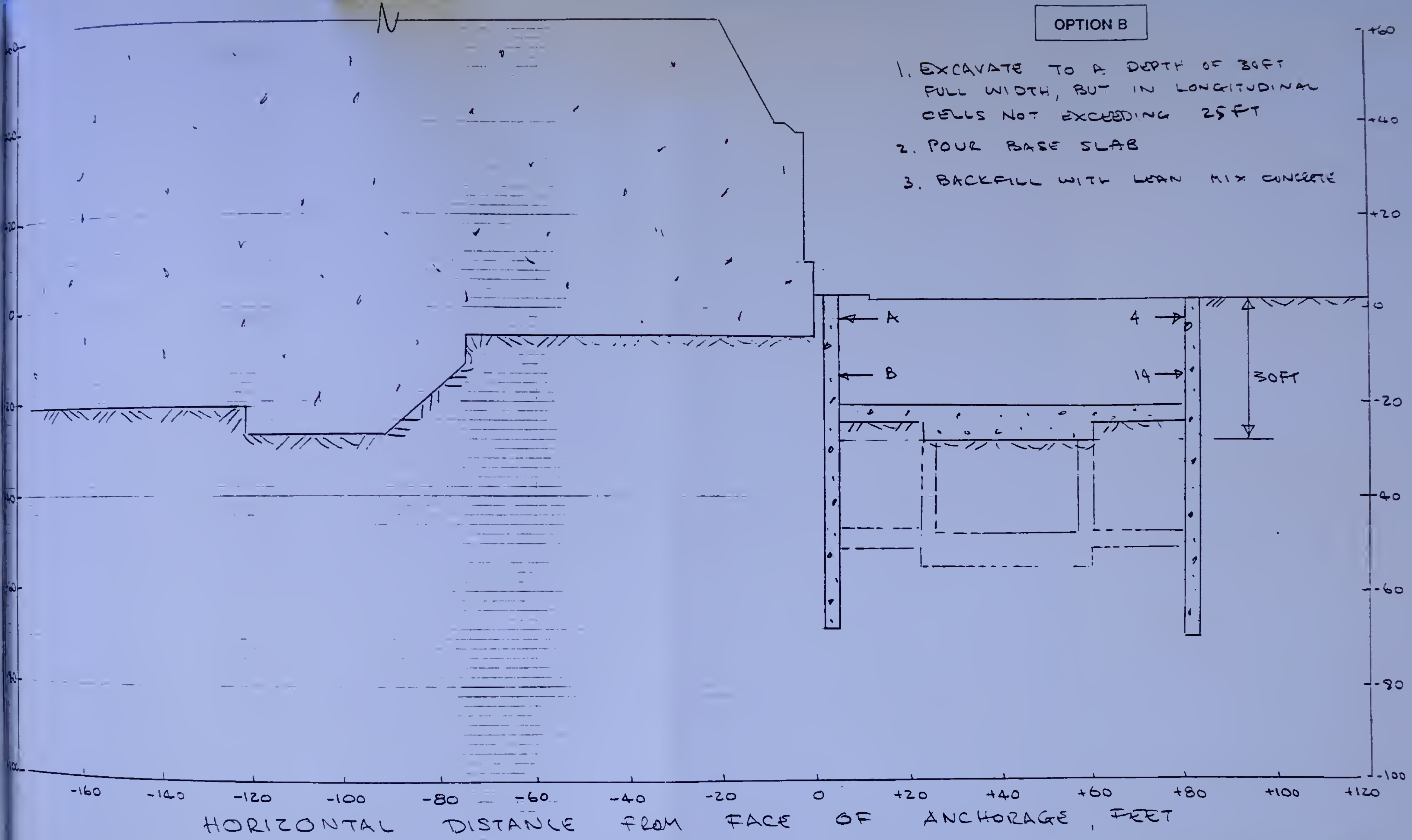
PLATE 6-32c

OPTION B



1. EXCAVATE TO A DEPTH OF 30FT FULL WIDTH, BUT IN LONGITUDINAL CELLS NOT EXCEEDING 25FT
2. POUR BASE SLAB
3. BACKFILL WITH LEAN MIX CONCRETE

STAGE I EXCAVATION, ALTERNATIVE 2A-C&C OPTION B, CUT-AND-COVER CONSTRUCTION: BAY BRIDGE ANCHORAGE AREA

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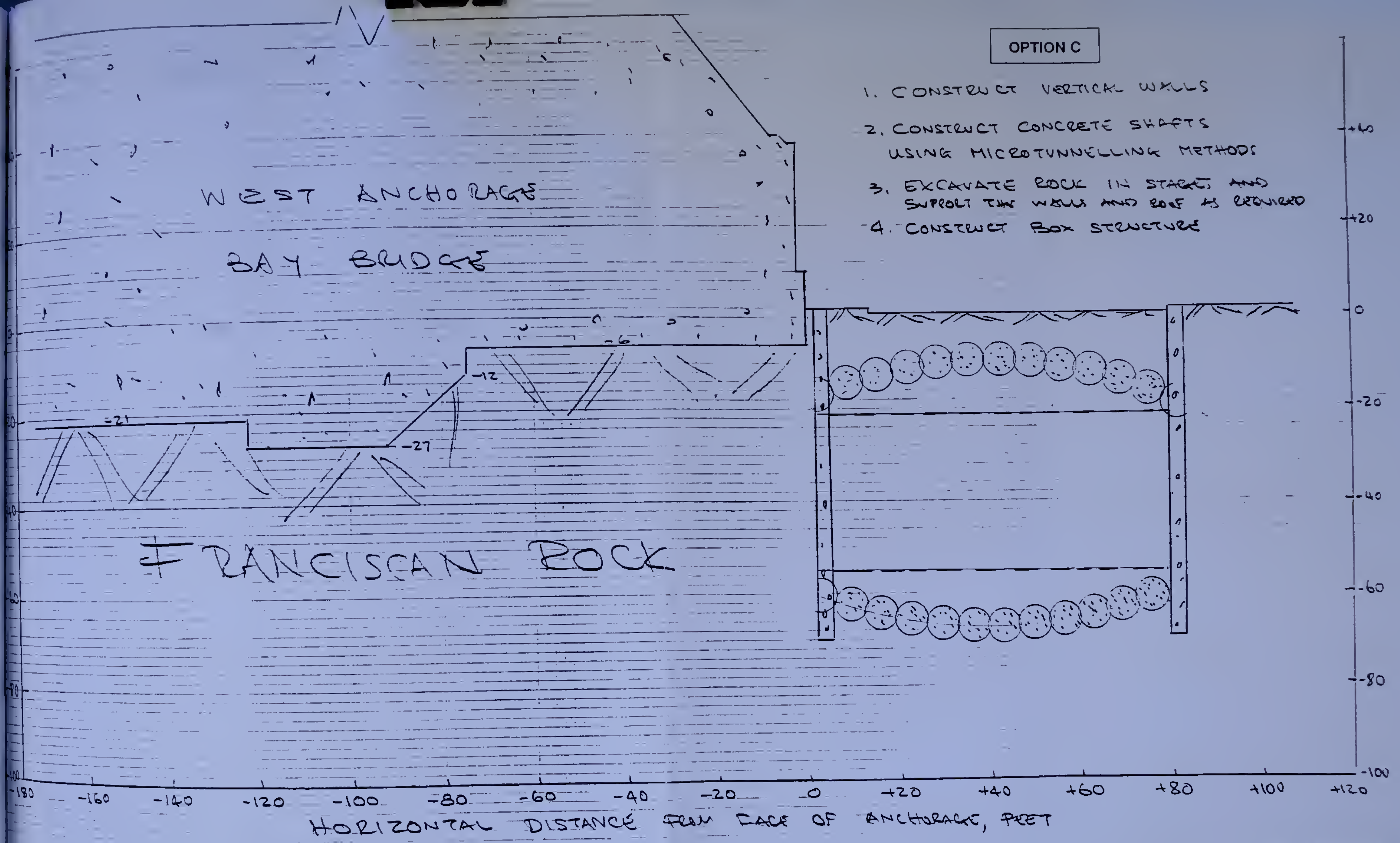
PLATE 6-33a



OPTION C

1. CONSTRUCT VERTICAL WALLS
2. CONSTRUCT CONCRETE SHAFTS USING MICROTUNNELLING METHODS
3. EXCAVATE ROCK IN STAGES AND SUPPORT THE WALLS AND ROOF AS REQUIRED
4. CONSTRUCT BOX STRUCTURE

EXCAVATION AT BRIDGE ANCHORAGE; ALTERNATIVE 2A-C&C; OPTION C

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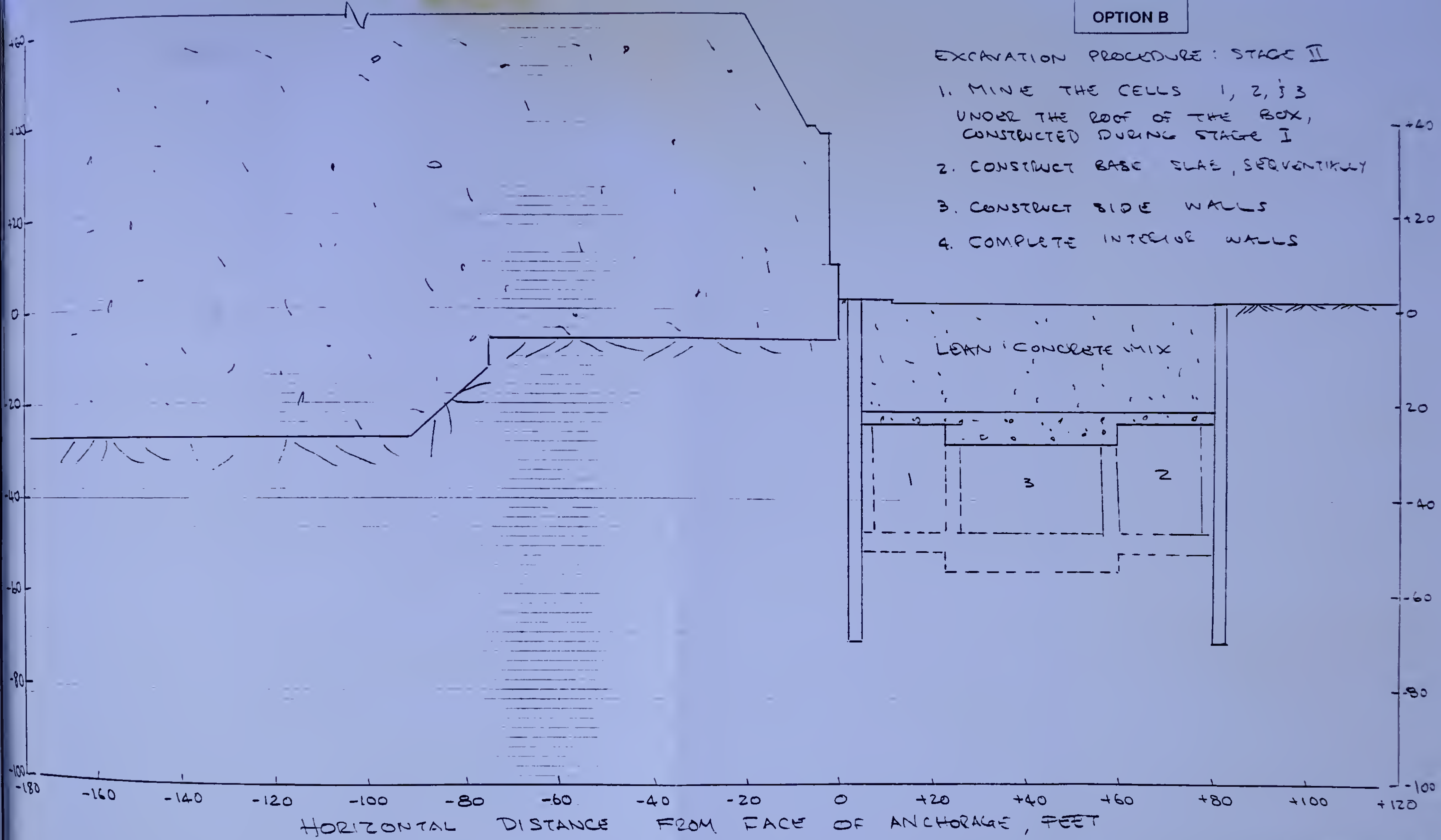
PLATE 6-34

OPTION B

EXCAVATION PROCEDURE: STAGE II

1. MINE THE CELLS 1, 2, & 3 UNDER THE ROOF OF THE BOX, CONSTRUCTED DURING STAGE I
2. CONSTRUCT BASE SLAB, SEQUENTIALLY
3. CONSTRUCT SIDE WALLS
4. COMPLETE INTERIOR WALLS



**STAGE II EXCAVATION, ALTERNATIVE 2A-C&C
OPTION B, CUT AND COVER CONSTRUCTION:
BAY BRIDGE ANCHORAGE AREA**

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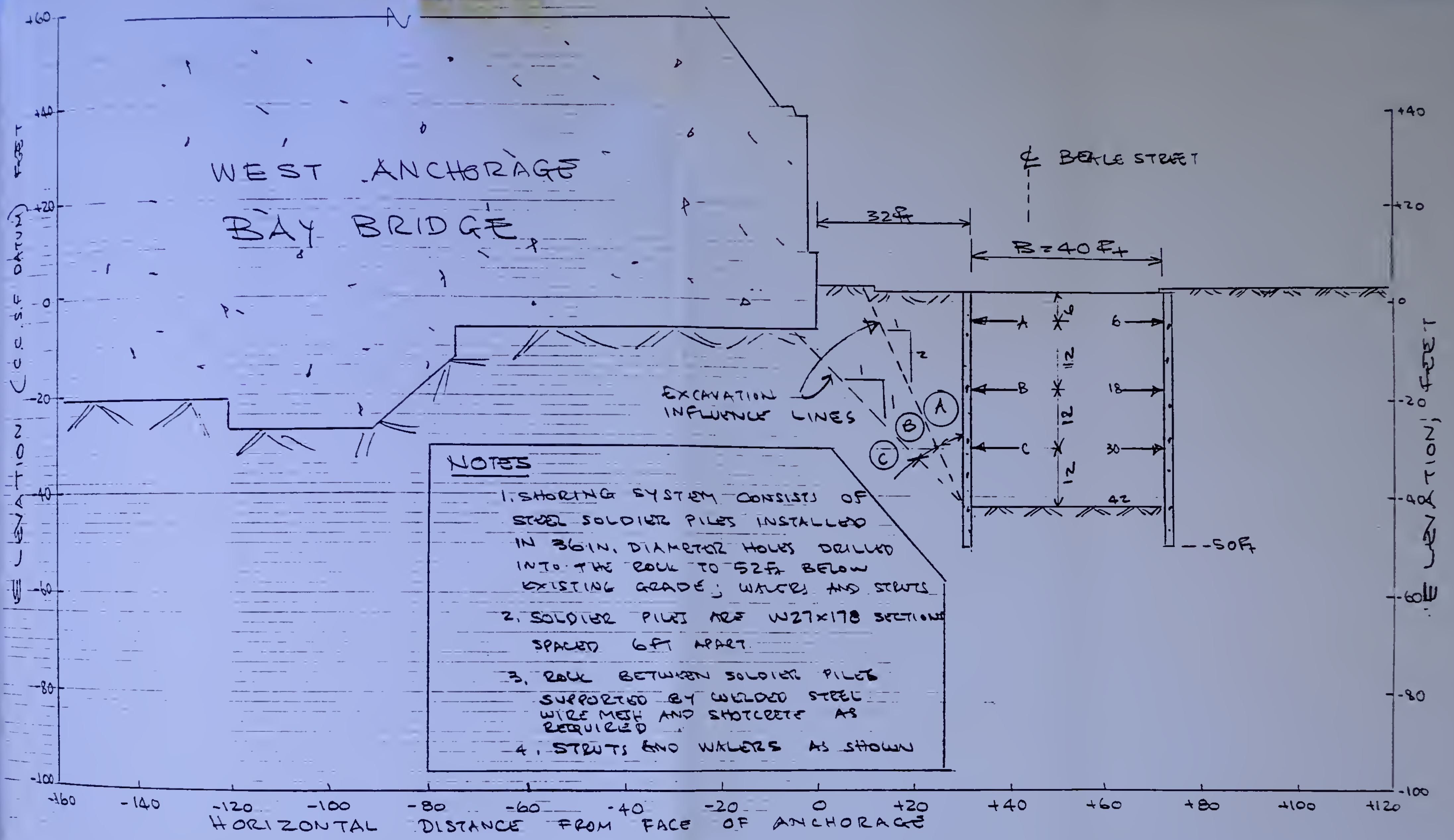
PLATE 6-33b



NOTES

1. SHORING SYSTEM CONSISTS OF STEEL SOLDIER PILES INSTALLED IN 36 IN. DIAMETER HOLES DRILLED INTO THE ROCK TO 52 Ft BELOW EXISTING GRADE; WALERS AND STRUTS
2. SOLDIER PILES ARE W27×178 SECTIONS SPACED 6 Ft APART.
3. ROCK BETWEEN SOLDIER PILES SUPPORTED BY WELDED STEEL WIRE MESH AND SHOTCRETE AS REQUIRED
4. STRUTS AND WALERS AS SHOWN

EXCAVATION AT BAY BRIDGE ANCHORAGE;
TWO TRACK ALTERNATIVE
(NARROWER AND SHALLOWER EXCAVATION)

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PLATE 6-35

7.0 TUNNELING

7.1 ALTERNATIVE TUNNELING ALIGNMENTS

Three options that involve tunneling along portions of the proposed alignments are currently being considered. The first option is for alternative 2A-MT. This alignment runs along Beale Street from Market Street to a point about 250 feet north of Harrison Street. At that point the alignment turns toward the southwest, passing west of the Harrison Bridge, cutting into Rincon Hill. Plate 7-1 shows a plan of the various alignments including 2A-MT and identifies the portions of each of the alignments that are planned to be constructed by tunneling. The tunnel segment starts at a point near Harrison Street and ends at the intersection of Third and Townsend Streets, forming a sweeping curve with a radius of about 3,000 feet. The total tunnel length for this alignment is about 2,300 feet. Plate 7-2 shows a subsurface section for the tunnel alignment together with subsurface soil and rock conditions.

The second option is for alternatives 3A and 3B. The tunnel segment for these two alternatives is about 1,750 feet long, starting at the north end near Folsom Street and terminating at the south end at a point approximately 300 feet north of Brannan Street. Plates 7-3a and 7-3b show the profiles for alternatives 3A and 3B together with subsurface conditions in terms of overburden soils and the type of rocks encountered in boreholes drilled along this alignment.

The third option involves a southward extension of the tunnels for alternatives 3A and 3B along a sweeping curve starting near Bryant Street and terminating near the intersection of Third and Townsend Streets. This is referred to as alternative 3AB-T. The total tunnel length for this alignment is approximately 3,000 feet. Plate 7-4 shows the tunnel profile and subsurface conditions for the portion of the tunnel south of Bryant Street.

The feasibility of tunneling was also investigated along a short portion of the alignment for alternatives 3A and 3B between stations 178+65 and 184+00. In this area the alignment passes under two historic buildings that must be preserved and protected from adverse construction effects. The alignment crosses under the buildings near the intersection of Colin P. Kelly Jr. and Townsend Streets on a 400-foot-radius curve. Plate 7-5 shows a cross-section of the subsurface conditions and the profile of this portion of the tunnels.

7.2 TUNNELING METHODS AND CONSTRUCTION ISSUES

As the subsurface sections on Plates 7-2 to 7-5 show, the tunnels will be excavated in the Franciscan Rock, although occasionally soft ground conditions may be encountered as for alternative 2A-MT in the vicinity of Bryant Street, and alternative 3A&B-T west of Third Street. As indicated in the site investigation report and as reiterated in Section 4.2.7 of this report, the rock is highly variable and consists mostly of highly fractured, sheared and weathered shales and graywacke sandstone. In some instances, the rock was found to be so intensely weathered that it has the characteristics of a very stiff to hard clay. However, there are inclusions of very hard sandstone, with RQD values in the range of 60-90%. These inclusions do not appear to be very frequent, and it is estimated that the amount of hard rock encountered over a given tunnel profile may range from 10-20% of the total volume excavated.

Under these circumstances, the use of tunnel boring machines to construct the tunnels is not considered feasible because of the great variability of the rock. The only viable option for excavating the proposed tunnels is by mining techniques. The most likely method that would be successful under these conditions involves partial face excavation with support of the face and each increment of excavation to maintain stability. Mechanical excavators such as roadheaders are likely to be proved an efficient excavation method within a good portion of the tunnel. Also, in some areas soft ground tunneling techniques may have to be used. However, it is recognized that when hard rock is encountered, it will probably be necessary to use limited blasting to loosen the rock before it is excavated.

During the course of the investigation of tunneling alternatives, we evaluated the options of constructing twin, horseshoe-type single-track tunnels, or a larger double-track tunnel. Because the method of excavation and support of the tunnel will require that each increment of excavation be secured before proceeding with the next stage, it is considered more efficient and probably less costly to construct a single double track tunnel, rather than repeat the phased excavation and incremental support process in two separate tunnel drives. During the first meeting with the Board of Special Consultants, the two alternatives were discussed and it was concluded that the engineering effort should focus on a single double-track tunnel. The Board's reports and recommendations are presented in the Appendix.

7.2.1 Tunneling in Soft Ground

As seen from the cross-section for Alternative 2A-MT and 3AB-T, the tunnel is likely to encounter soft ground conditions that will require mixed-face tunneling. It is also evident that, along

a significant portion of alignment 2A-MT as well as along the southern end of Alternative 3B, the rock cover would be relatively thin, as little as a few feet. Typically, the rock is highly weathered and highly fractured, especially near the top of the rock. In some areas the rock may be so soft that the only way to secure the stability of the face and the roof is by using soft ground tunneling techniques. Other approaches involve identification of soft zones during the final design stage by an appropriate exploration program and treating extensive zones of soft ground by appropriate soil improvement methods. Conditions such as those encountered near Bryant Street along alignment 2A-MT could be treated in advance of tunneling by jet grouting or other appropriate methods.

Notwithstanding the efforts to treat areas of known weak materials, it may be impossible to treat every weak zone in advance of construction, either because it may not be possible to identify all of the weak areas or because access from ground level may not be available to carry out the required treatment program.

In view of the potential for encountering unanticipated zones of soft ground during tunneling, and in view of the inevitable fact that mixed-faced conditions would prevail along most of the alignment, the Board of Consultants recommended that provisions be made at the outset of tunneling to make available the necessary equipment and personnel for compressed air tunneling. These provisions would make it possible to mobilize quickly, if needed, without causing excessive delay in the performance of the work.

7.2.2 Dewatering

The tunnels will be constructed below the groundwater table. In portions of the rock, especially where poor quality (low RQD), sheared and fractured sandstone is encountered, the mass permeability of the rock might be very high, resulting in significant groundwater flow into the tunnels. For the partial excavation procedure to be feasible, it is essential to control the groundwater during tunneling. This can be accomplished by dewatering to lower the groundwater along the tunnel alignment below the tunnel invert. Wells would have to be installed along the alignment, extending below the invert of the tunnel, and dewatering would have to start at least 60 days and preferably 90 days before the tunnel heading reaches a particular point. Installation and operation of the wells will require the cooperation of property owners along the alignment. Where the alignment passes under existing buildings, access to install and operate the wells may not be available. If dewatering cannot be carried out externally, it may be necessary to lower the groundwater from inside the tunnel using horizontal drains installed in front of the tunnel heading to predrain the rock. Such a procedure would be time consuming, difficult and extremely costly.

Dewatering can cause serious problems, adversely affecting the foundations of existing structures. Where soft soils are present, such as the Bay Mud, dewatering can cause consolidation and settlements. These settlements could impact existing structures directly, or indirectly by imposing significant downdrag loads on pile foundations. Other problems that could be caused by dewatering involve the exposure of old wood piles to air, which could result in rapid decay and adverse impacts on the buildings supported by the wood piles.

7.2.3 Other Construction Issues

The use of compressed air with an open face shield raises a number of concerns. Most significant among them is the migration of air from the tunnel face as well as from the space around the shield and liner. Air carrying with it gases that may be released by the groundwater could infiltrate into adjacent buildings resulting in significant environmental and health hazards for the occupants of adjacent structures. The loss of air from the tunnel is very difficult to mitigate.

Because the ability to avoid instability and ground loss will depend heavily on the alertness and quick response of tunneling crews, the risks of tunneling involve a significant human factor. Even when skilled and experienced tunnelers are available, the tunneling process is still vulnerable to temporary lapses in the level of skill and care required to maintain control in rapidly changing ground conditions. What is particularly difficult about the proposed construction is that there is little cover over the tunnel along a substantial portion of the proposed rock alignments. Instability of the face or roof could propagate easily to the surface, causing damage to buildings and other facilities.

7.2.4 Excavation Staging

Plate 7-6 shows a typical cross-section of the double-track tunnel. The rock cavity, on final excavation, will be approximately 45 feet wide, about 33 feet high at the middle, and 23 feet high along the sides. The roof is a flat arch 33 feet in radius.

The excavation procedure recommended by the Board of Special Consultants consists of the following incremental steps:

1. A central drift and two wall drifts are first excavated and supported in the tunnel heading. More than one drift could be excavated simultaneously, although the excavation process would have to be staggered. From inside the drifts, grouting could be performed as required to stabilize the rock over the roof and outside the

sidewalls of the final opening. Rockbolts, if required, could also be installed from the drifts.

2. Once the drifts have been completed and their stability secured with temporary supports such as steel ribs, lagging and/or wire mesh with shotcrete and rockbolts, vertical supports are installed in the center of the middle drift and on the far wall of each of the side drifts. These vertical supports are used to prop the steel ribs during the next excavation stage.

3. The heading is excavated in two stages (designated 4 and 5 in Plate 7-6) and steel ribs are erected to support the roof. It must be recognized that, because of the poor quality rock, the steel ribs should be installed as soon as sufficient rock is excavated to make installation of the ribs possible. In lieu of excavating the full areas associated with steps 4 and 5, it is recommended that the top portion of each area be excavated first to allow rapid installation of the ribs supporting the roof.

4. Once the heading has been excavated and the stability of the face and roof secured, the bench can be excavated in stages. As excavation of the bench progresses, the walls of the cavity are supported by rock anchors and/or grouting the rock mass behind the wall. Grouting behind the walls of the tunnel can be carried out before the bench is excavated as shown on Plate 7-7.

There is obviously a question whether the bench will be excavated in two full-width lifts or two half-width, full depth passes. These are details that will have to be developed during final design, after additional exploratory data have become available, and undoubtedly will have to be tried and evaluated during tunneling.

7.3 DESIGN CONSIDERATIONS

From the discussion of the tunneling method presented in the previous section of this report, it is clear that the face as well as the roof and sidewalls of the tunnel would have to be secured in an incremental manner as excavation advances. Because of the flat shape of the arch (which is necessitated by the requirement to have as much cover as possible), the stresses in the steel ribs, as well as the thrust loads at the wall plates, will be significant, and result in heavy steel sets for roof support. Because of the poor quality of the rock, for design purposes it should be assumed that the rock will behave as a dense/stiff soil. Given that most of the alignment for alternative 2A-MT as well as a good portion of alternatives 3A and 3B would have fairly shallow cover, we recommend for

preliminary design purposes that the roof load be assumed to be the total overburden stress, as a uniform vertical pressure acting over the tunnel roof. Exceptions to this are short sections of alternatives 3A and 3B between station 195+00 and 202+00. Smaller loads could be used in these areas, but such refinement should be left for subsequent phases of the design process.

Because of the poor quality of the rock, the horizontal pressures to be resisted by the side wall supports would be high. Assuming that during construction the groundwater table would be lowered below the tunnel invert, there would be no hydrostatic forces on the temporary support system. However, we do not recommend that the groundwater levels be lowered permanently by provision of drainage around the tunnel; such a lowering of the groundwater table could have severe effects on existing structures. In addition, the environmental impacts from permanent groundwater lowering are likely to be unacceptable. Waterproofing will have to be provided and the permanent lining should be designed to resist the earth and water pressures acting on the permanent lining.

During construction the horizontal stresses on the vertical shoring system supporting the walls may be assumed as a fraction of vertical stresses, by the equation:

$$\sigma_h = K\sigma_{vo}$$

where σ_h is the total horizontal stress,
σ_{vo} is the total vertical effective stress, and
K is an empirical earth pressure coefficient.

The value of the coefficient K may be taken as equal to 0.4. The basic assumption in selecting the value of K is that the system will have to be designed rather conservatively in view of the high variability of the rock. Further studies and analyses may allow reduction of this factor, especially where good quality rock is encountered.

For the design of the permanent lining, the hydrostatic pressures will have to be added to the earth pressures in accordance with the following equation:

$$\sigma_h = K\bar{\sigma}_{vo} + U_w$$

where $\bar{\sigma}_{vo}$ is the vertical effective stress, which is equal to the total overburden stress minus the water pressure, and

U_w is the hydrostatic water pressure.

The above equation may also be expressed as:

$$\sigma_h = K(\sigma_{vo} - U_w) + U_w$$

The earth pressure recommendations for the design of the temporary support system and the design of the permanent lining are presented on Plate 7-8.

The design criteria will have to be modified as required when additional exploration data become available during the design stage. Furthermore, because of the highly variable nature of the rock, the design may have to be modified during construction to suit the conditions encountered. In this regard, the information developed during the construction of the drifts shown on Plate 7-6, and the experience accumulated in terms of the required support for the face and side walls, will help significantly in refining the design of both the temporary and permanent supports.

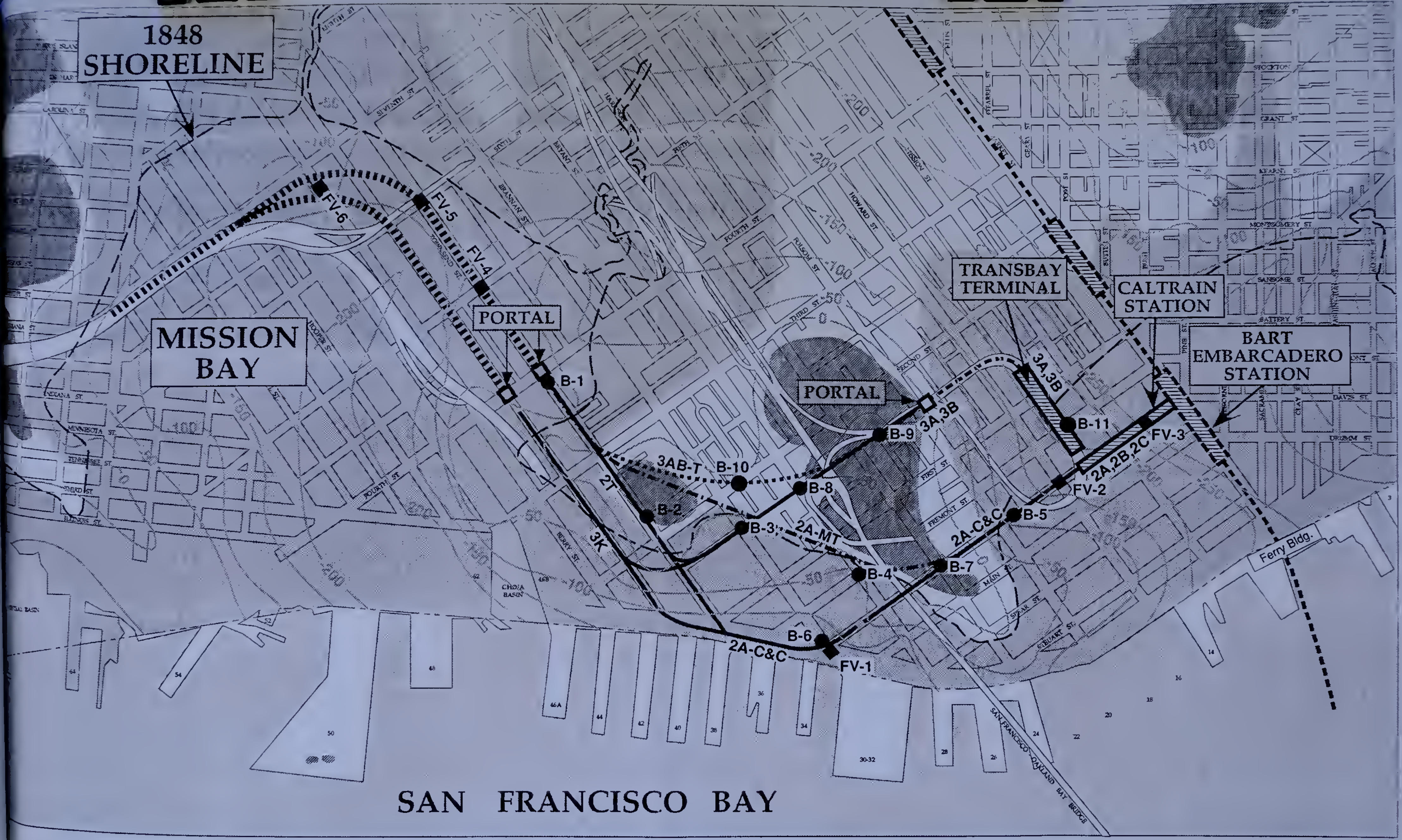
7.4 CONCLUDING REMARKS

From the earlier discussions regarding the potential difficulties during tunneling, it is obvious that the proposed tunneling alternatives present a significant risk to structures and other facilities under which the tunnels will pass. In particular, the two alternatives that involve long tunnels (2A-MT and 3AB-T) would pass under a relatively large number of significant structures. It is clear that the risks associated with these two alternatives are much greater, as compared to the risks associated with the two alternatives (3A and 3B) that involve shorter tunnels between Folsom and Brannan Streets. Another important difference between the long and short tunnels is that the rock cover over the tunnels for alternative 3 is significantly greater than for the long tunnel alignments (see Plates 7-3a and 7-3b). In areas of deep rock cover, instability of the face or the roof is unlikely to propagate to the surface as a catastrophic ground loss.

Another important advantage associated with the short tunnels for alternative 3 is the fact that the tunnels pass under only three buildings located in the area between Bryant and Brannan Streets. These buildings are relatively old, single or two-story structures. If necessary, these buildings could be evacuated while the tunnel heading passes under these buildings to eliminate the risks to health and safety of the occupants. Damage that may develop due to tunneling could be repaired at relatively modest costs.

In conclusion, as far as tunneling is concerned, the most favorable alignment is the one associated with the short tunnels. Obviously, alternative 3B, which involves the deeper tunnel profile, is preferable to alternative 3A.

Tunneling under the historic buildings near the intersection of Colin B. Kelly Jr. and Townsend Streets is feasible, but does not appear to be the most effective solution. It is our opinion that, in lieu of tunneling, cut-and-cover construction methods may be used. What would be required is that the buildings be evacuated, and the walls reinforced and underpinned to carry the loads beyond the limits of the excavations. Following these preparatory/precautionary measures, excavation can be carried out using the methods and procedures previously described in Section 6.0.



KEY

●	B-5	Boring Location
■	FV-3	Field Vane Location
—·—·		Tunnel Route Of Alternative 2
·······		Tunnel Route Of Alternative 3
		Bedrock Contours (feet)
▨		Fill
▩		Rock Outcrop

PLAN OF TUNNELING ALTERNATIVES

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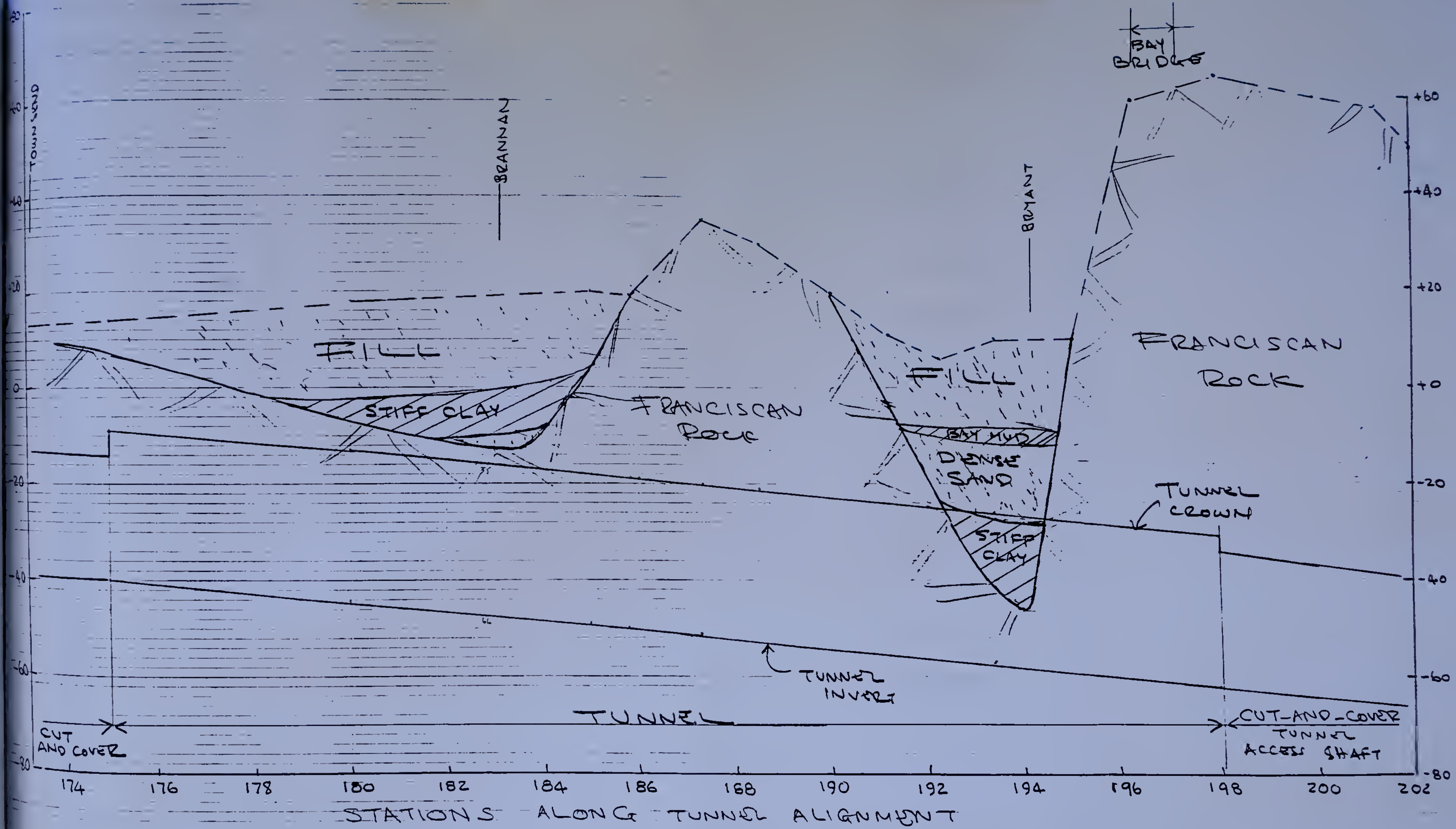
PLATE 7-1



TUNNEL PROFILE: ALTERNATIVE 2A-MT

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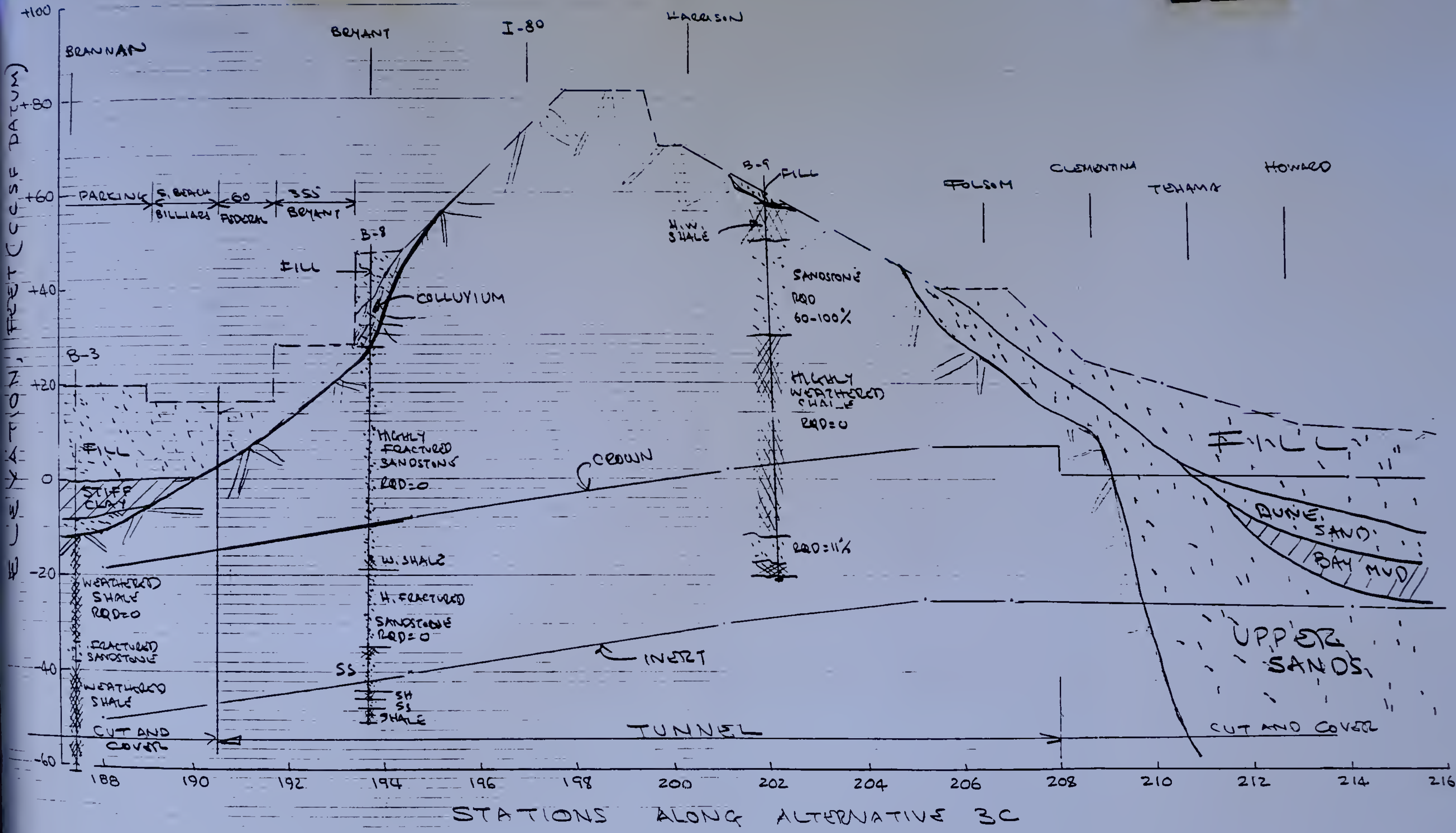
PLATE 7-2



TUNNEL PROFILE: ALTERNATIVE 3B-T
(FOLSOM TO BRANNAN)

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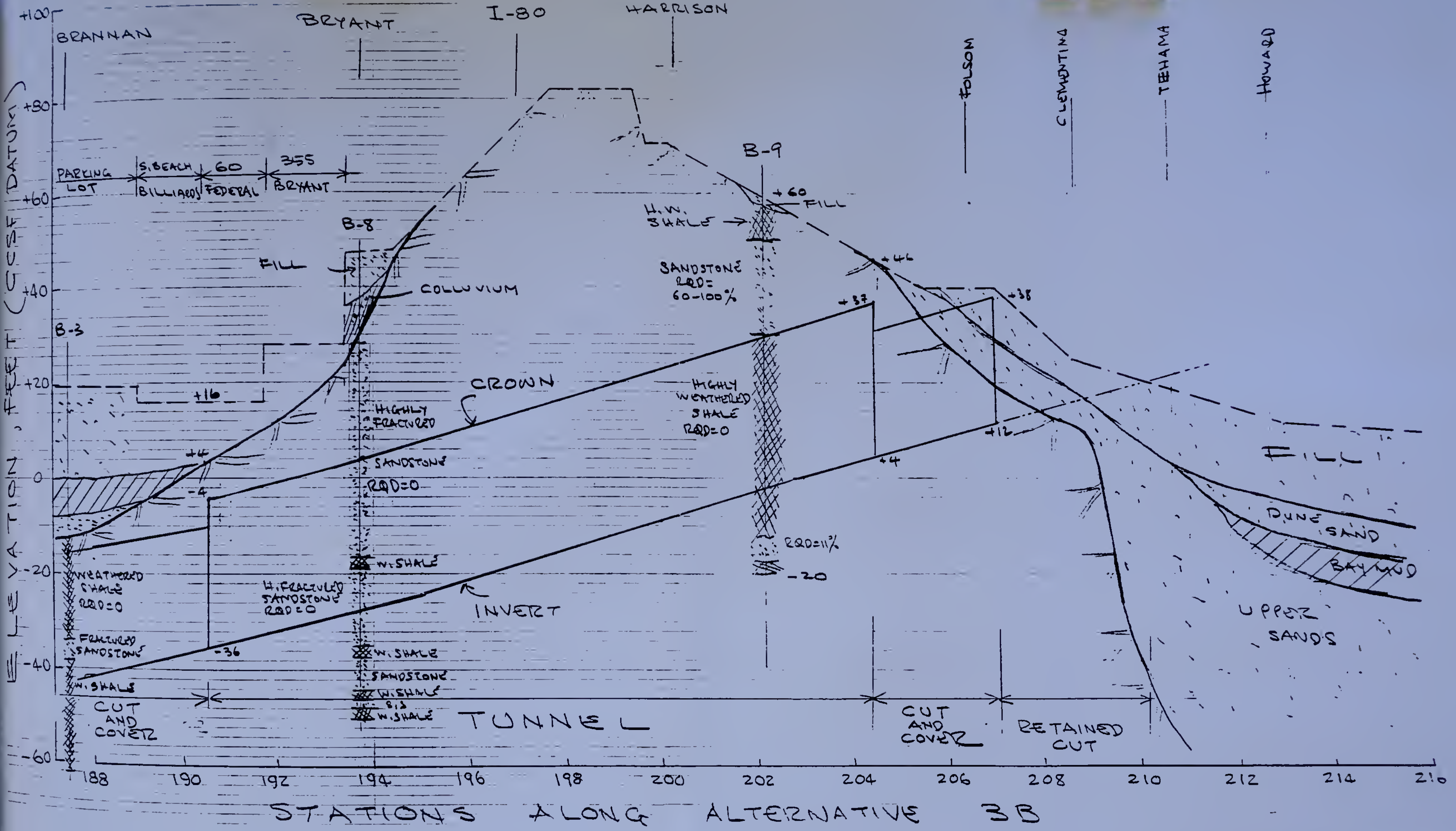
PLATE 7-3a



TUNNEL PROFILE: ALTERNATIVE 3A-T (FOLSOM TO BRANNAN)

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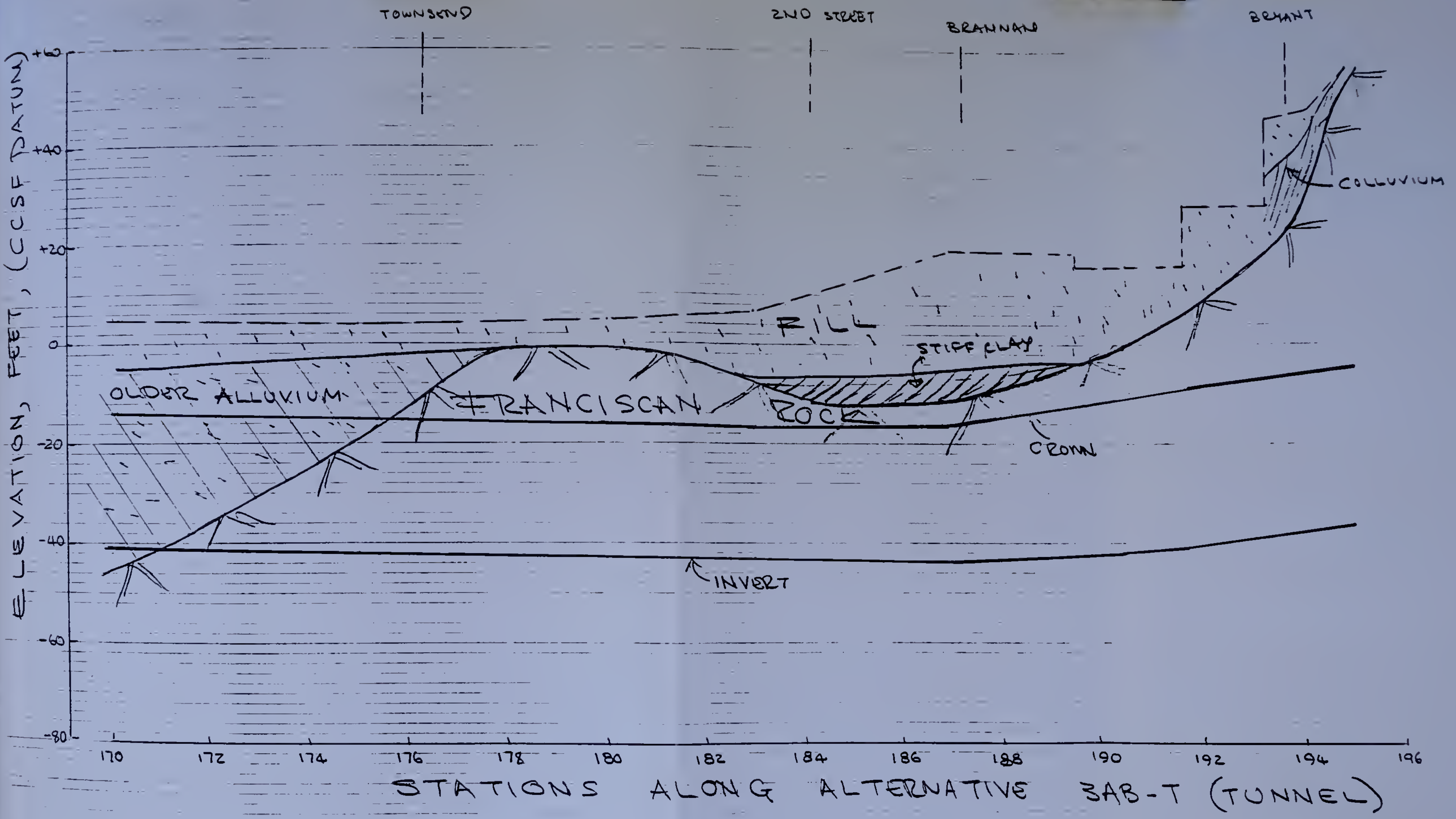
PLATE 7-35



TUNNEL PROFILE: ALTERNATIVE 3AB-T
(LONG TUNNEL: BRYANT TO TOWNSEND)

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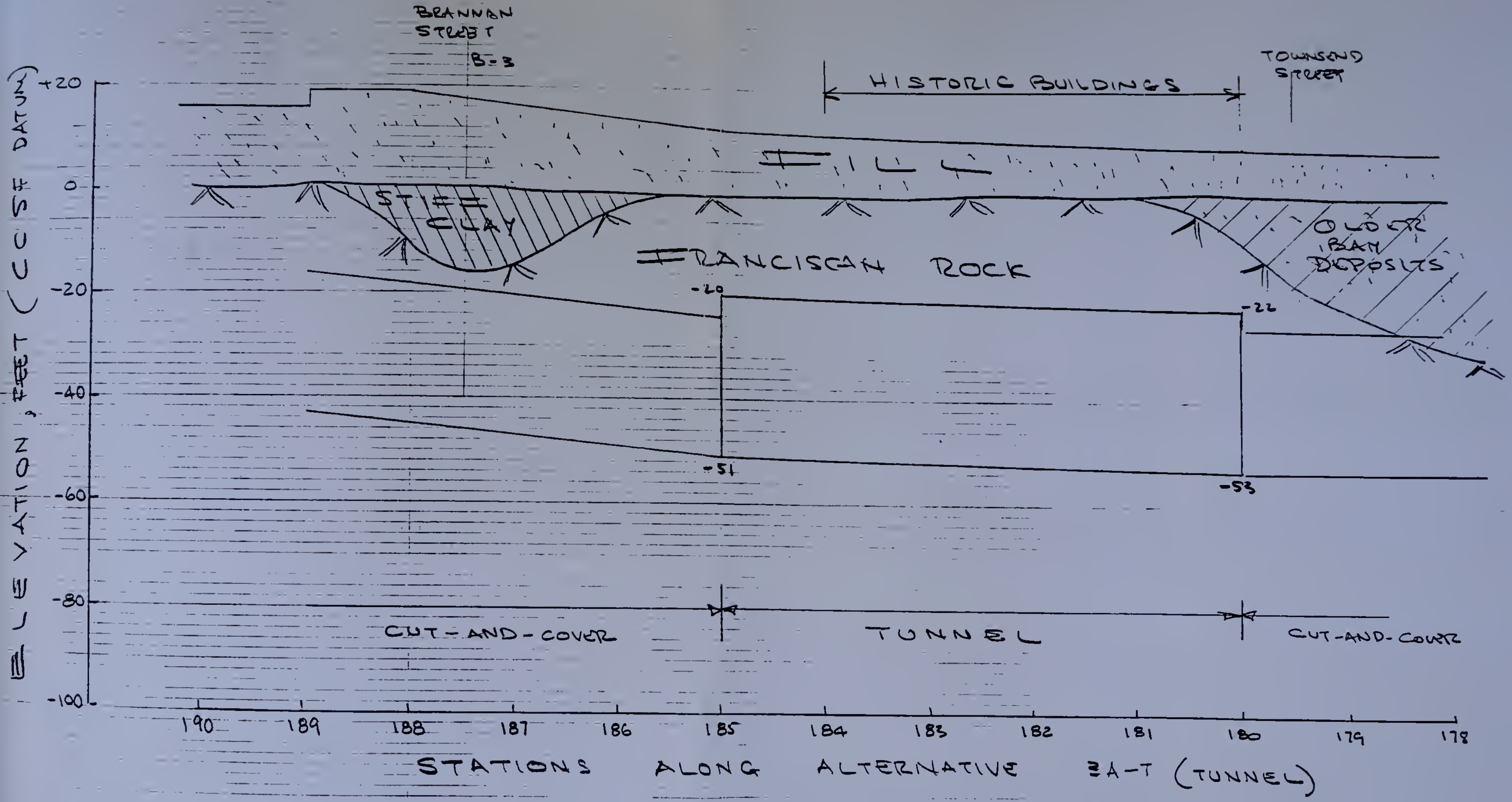
PLATE 7-4



PARTIAL TUNNEL PROFILE: ALTERNATIVE 3A-T (TUNNEL UNDER HISTORIC BUILDINGS)

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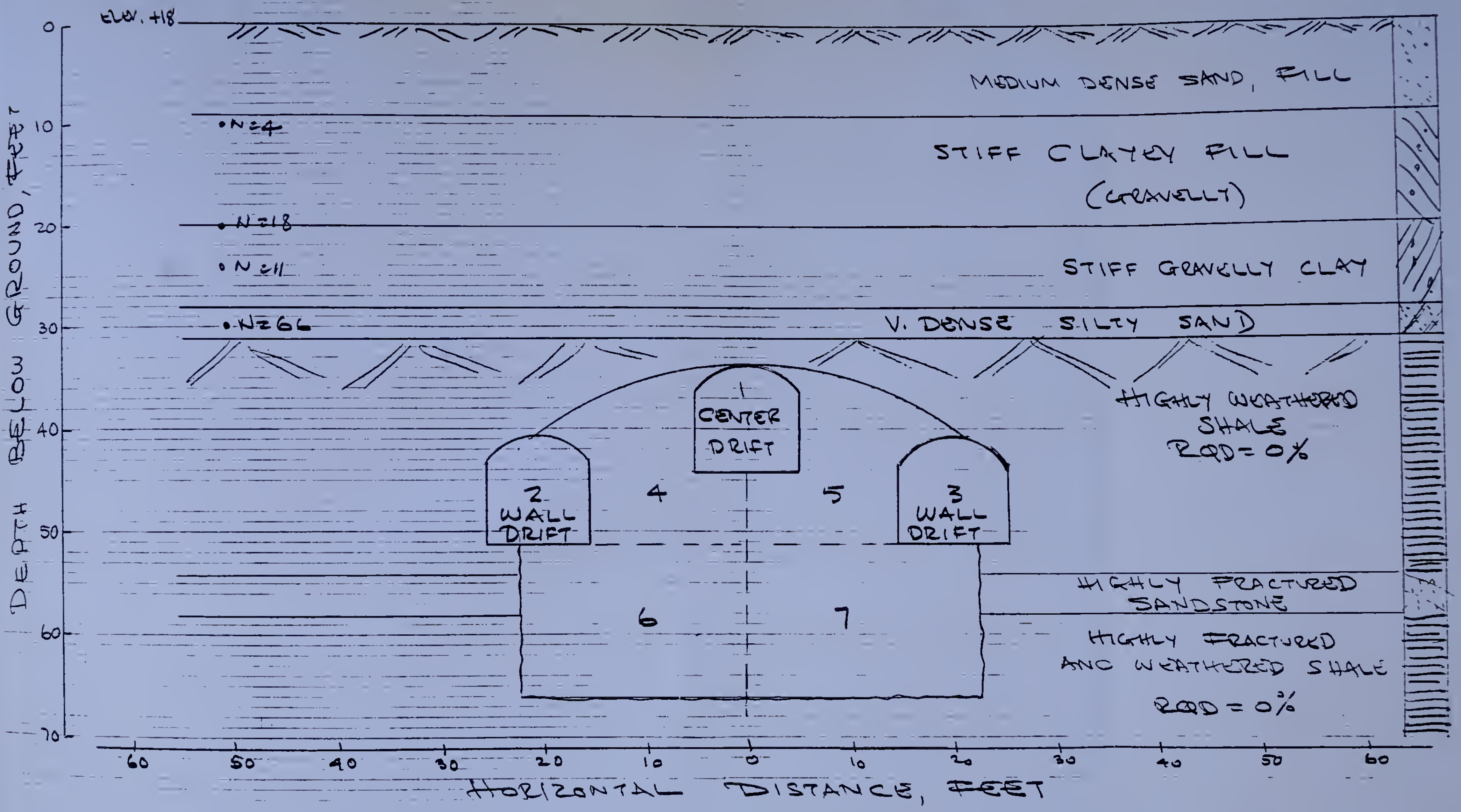
PLATE 7-5



PARTIAL FACE EXCAVATION
DOUBLE TRACK TUNNEL

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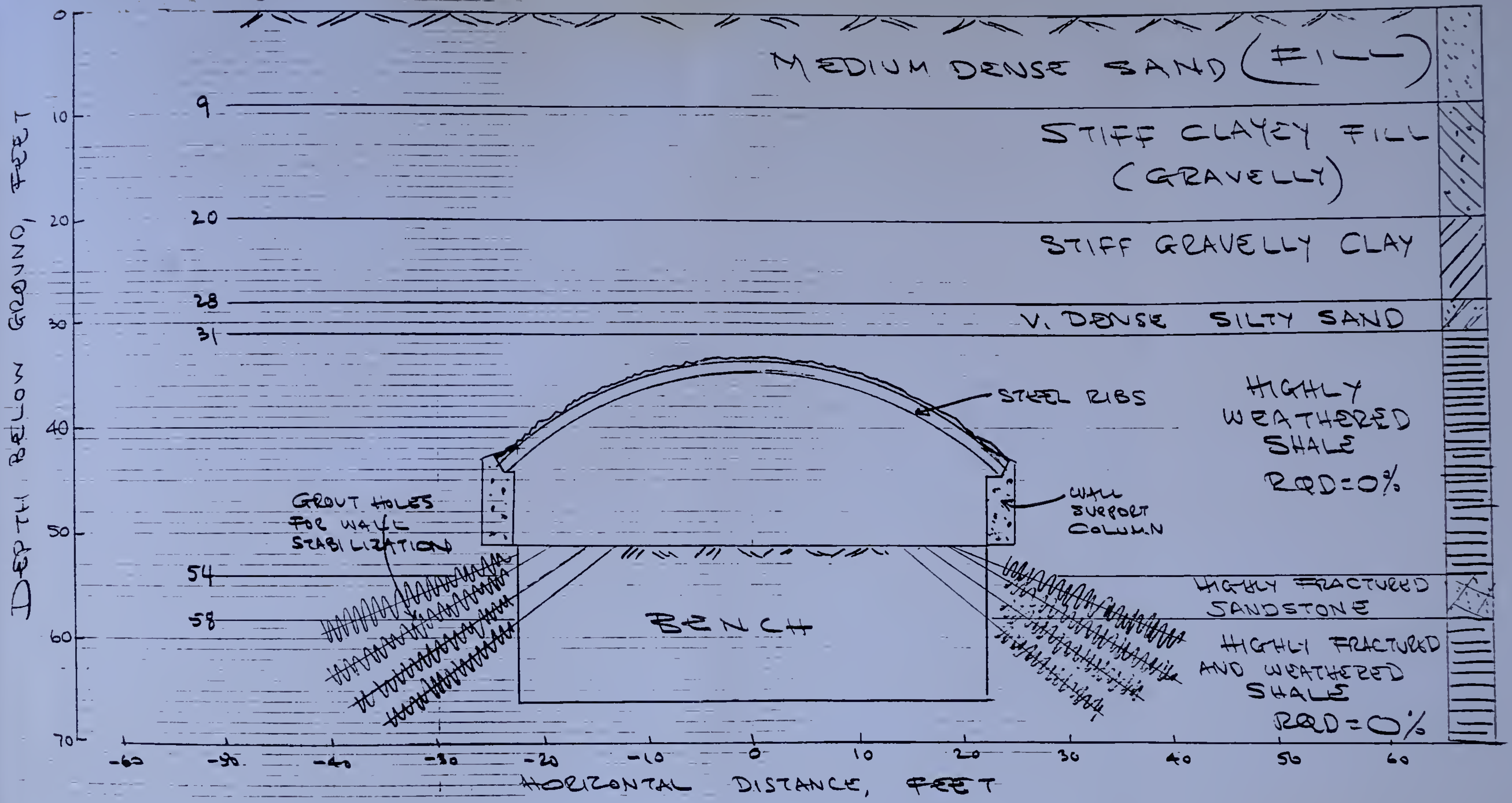
PLATE 7-6



ROCK STABILIZATION BY
GROUTING BELOW BENCH

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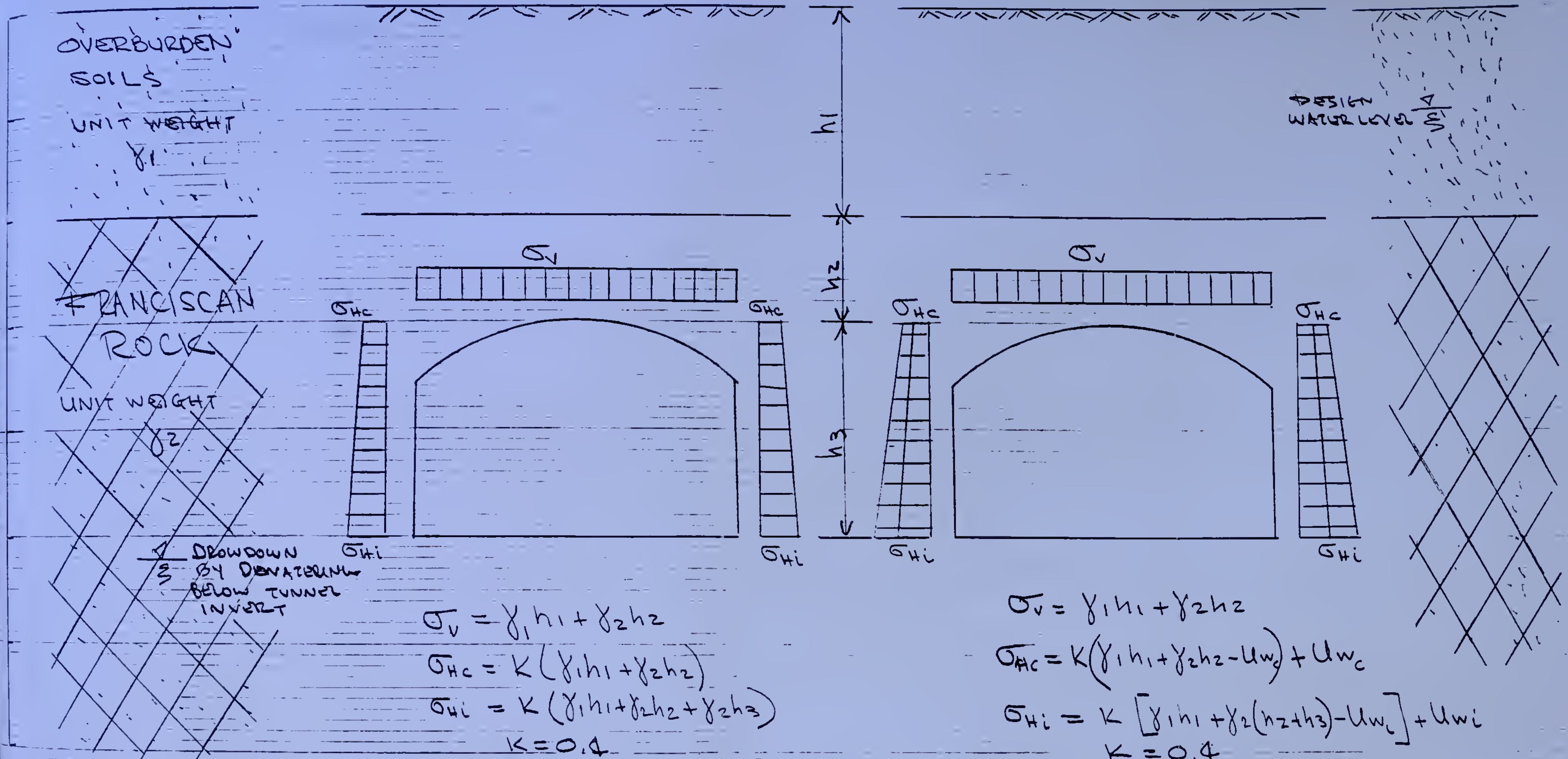
PLATE 7-7



$$\sigma_v = \gamma_1 h_1 + \gamma_2 h_2$$
$$\sigma_{Hc} = K(\gamma_1 h_1 + \gamma_2 h_2)$$
$$\sigma_{Hi} = K(\gamma_1 h_1 + \gamma_2 h_2 + \gamma_2 h_3)$$
$$K = 0.4$$

(a) PRESSURE DIAGRAMS FOR DESIGN OF TEMPORARY SUPPORTS

$$\sigma_v = \gamma_1 h_1 + \gamma_2 h_2$$
$$\sigma_{Hc} = K(\gamma_1 h_1 + \gamma_2 h_2 - U_{w_c}) + U_{w_c}$$
$$\sigma_{Hi} = K\left[\gamma_1 h_1 + \gamma_2 (h_2 + h_3) - U_{w_i}\right] + U_{w_i}$$
$$K = 0.4$$

(b) PRESSURE DIAGRAMS FOR DESIGN OF PERMANENT SUPPORTS

PRESSURE DIAGRAMS FOR THE DESIGN OF TUNNEL SUPPORTS AND PERMANENT LINER

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PLATE 7-8

8.0 FOUNDATION SUPPORT

In most instances, the excavations are deep enough so that the base slab of the permanent box structures would be founded on competent foundation soils. In such instances, unless there are significant net hydrostatic uplift forces, the box structures can be designed as a mat foundation without the use of piles.

There are two notable exceptions, where the base slabs of the permanent structures are founded in the Bay Mud. The first involves the foundation of the terminal for alternative 3C. The base slab is planned to be constructed at elevation -30 feet. Bay Mud is expected to be found below this elevation. Moreover, the thickness of mud below the base slab will vary substantially. Because the mud is very soft and compressible, and because the thickness below foundation level varies considerably, we recommend that the terminal be supported on piles driven to penetrate into the very dense upper sand strata. The lengths of the piles would vary from 30 to 40 feet. Prestressed concrete piles, 14 inches square, driven to a driving resistance of 10 blows per inch, could be designed with an allowable load of 150 tons. During final design, other alternatives would be explored including the use of a deeper basement that might eliminate the piles. The extra cost of the deeper excavation would be offset at least partially by the savings realized from eliminating the piles. The result would be the extra space from a deeper basement (perhaps one more level of space) at relatively little extra cost.

The second case, which involves deep Bay Mud below the base of the permanent box structures, is in the area between Fourth and Seventh Streets along Townsend Street. The presence of the deep Bay Mud raises a concern with the post-construction settlements of the box, unless the permanent structures are supported on piles. Assuming that some post-construction settlement can be tolerated, piles may not be necessary. This is because the Bay Mud appears to be somewhat overconsolidated, at least along some portions of this segment. For preliminary design purposes, we recommend that the box structures be designed so that they can be tied into the soldier piles for the temporary shoring system. In this way, they can derive vertical support from the steel H-piles. Of course, the base slab must be designed so that it can span between the soldier piles of the two walls.

During final design, additional studies will be required to evaluate further whether pile foundations may be required in this area. The primary focus of the additional studies would be to evaluate the preconsolidation characteristics of the Bay Mud and estimate post-construction settlements so that an evaluation can be made about whether the estimated settlements can be tolerated by the facility.

9.0 IMPACTS OF EXCAVATIONS AND TUNNELING

The processes of excavation and tunneling involve removal of significant volumes of soil and readjustment of the in-situ stresses to new equilibrium values. As a result of the changes in the stress system, ground deformations develop during excavation and tunneling. The objective of this section is to review briefly the types of deformations, likely magnitudes for the anticipated construction, and potential impacts on existing structures and other facilities, that are within the zone of influence of the planned excavations and tunnels.

9.1 GROUND MOVEMENTS CAUSED BY EXCAVATIONS

Plate 9-1 shows a conceptual model of the development of ground deformations during excavation. The removal of the soil within the excavation causes the shoring walls to move inward as shown diagrammatically on the left-hand side of Plate 9-1. As a consequence of the inward wall movements, the ground retained by the excavation walls settles. The magnitude of the lateral deflections and settlements, as well as the zone of influence of the ground movements, vary significantly depending on the ground conditions, the construction procedures employed, and the stiffness of the shoring system. As an example, if the excavation is carried in soft mud that extends to great depth below excavation level, the ground movements would extend deep below the excavation and to a considerable distance behind the excavation, as shown on Plate 9-1. On the other hand, if competent strata such as the Colma sand or rock are present below excavation level, the depth of the movements below the base of the excavation would be relatively small, and the distance behind the excavation that is affected by settlements would also be small, as compared to the case of deep soft soils. Stiff systems such as thick concrete slurry walls and closely spaced struts would result in smaller movements than if less stiff walls and/or widely spaced struts are used.

Regardless of the ground conditions and the system used to support the excavations, some lateral movement and settlement should be expected. The shoring recommendations presented in section 7 of the report are intended to minimize the settlements and lateral deflections caused by the excavations. The effects of excavations in the various areas that could impact existing structures are reviewed below.

9.1.1 Excavations Involving Soft Bay Mud

The portions of the excavations for Alternative No. 2 that are in the old Yerba Buena Cove area and the South Beach area would be excavated in soils, including relatively thick deposits of Bay Mud. In the Mission Bay Area the behavior of the excavations will be dominated by the Bay Mud,

ecause it extends to significant depth below the base of the planned excavations. In these areas, here are major structures that are very close to the excavations.

The shoring systems recommended in section 7 take into account the potential effects of ateral deformations and settlements on the existing structures. Where necessary, special measures ave been developed to minimize the movements and keep them within tolerable limits. As an xample, in areas where the excavations extend below the tips of the piles supporting adjacent tructures, internal struts constructed using specialized techniques before excavation begins have been ecommended. Other measures include subdividing the excavation into small cells to control ground ovements. The objective of the proposed measures is to provide shoring systems that are capable f controlling building settlements to values less than one inch. The current plans to limit the depth f the excavations to 55 feet or less so that the tips of pile foundations supporting adjacent structures re below the base of the excavations is a very significant factor in limiting the settlements of adjacent uildings and reducing the costs associated with special measures that would otherwise be needed o control settlements within tolerable limits

In the area along the Embarcadero roadway, the thickness of the Bay Mud is small and the xcavations would extend below the Bay Mud In such cases, the Bay Mud is not the dominant actor in terms of ground deformation. Nevertheless, existing buildings and other facilities are located ithin a few feet of the excavation walls in this area. The ground movements caused by the xcavations can be controlled to one inch or less and the settlements of pile-supported buildings ould be limited to a fraction of one inch. However, the tracks of the Muni Metro railway and other tilities supported near the ground surface would settle with the ground. It is not considered conomically viable to design the shoring system in this case to control movements to values less than ne inch. It is probably more cost-effective to remedy the problems caused by settlements rather than o try to eliminate the settlements.

In the Mission Bay Area between Fourth and Seventh Streets, the excavations are generally bout 42 feet deep. Because there is considerable depth of Bay Mud below the base of the xcavations, lateral deflections and settlements could be significant.

The proposed shoring system for the Mission Bay Area is expected to result in maximum round movements on the order of 2 inches. Such movements would be tolerable because the xcavations are typically located more than 30 feet from any significant buildings. Exceptions to this re the on and off ramps for the I-280 Freeway and Sixth Street. At this location, the proposed xcavation will be within a few feet from one of the columns supporting the ramp. It is our valuation that the settlement of the ramp would be very small because the columns are supported

on piles penetrating deep below the base of the proposed excavations. However, the lateral deflections of the ground would apply lateral loads on the pile foundations. The magnitude of the loads and their effects on the piles should be evaluated during final design, but they would not pose a serious concern for the project.

Excavations penetrating in rock would also cause some lateral deformations and settlements. However, if the shoring systems recommended in section 7 of this report are used, and if good workmanship is used to construct the excavations, the movements caused by excavations in rock should be small. The zone of influence of these movements should not extend more than ½ to 1.0 times the depth of the excavation from the edge of the excavation.

Dewatering associated with the excavations in rock could have much more important effects in terms of settlements than the excavation process. This is because the excavations are fairly deep at some locations and would involve lowering the groundwater table by as much as 50 or 60 feet to keep the excavations dry. The effects of dewatering could spread over a long distance from the excavations and could cause consolidation of the Soft Bay Mud, resulting in settlements. It is very difficult at this stage to evaluate accurately the potential settlements due to dewatering or their impacts on existing facilities and to determine whether mitigation measures should be implemented. The dewatering is an issue that would require careful evaluation during the forthcoming design stages of the project.

Dewatering could also affect pile foundations, because lowering the groundwater table might expose old wood piles to oxygen and bacteria and could cause deterioration of the piles. Again, this issue will require careful evaluation during the detail design phase of the project.

9.2 GROUND MOVEMENTS CAUSED BY TUNNELING

Tunneling involves the removal of earth materials to create a cavity below ground surface. The removal of the soil or rock causes a readjustment in the ground stresses, which in turn results in ground movements reflected as settlements at the ground surface and as lateral movements on either side of the tunnel. Plate 9-2 shows the shape of the distribution of settlements at the ground level in response to tunneling. The maximum settlement occurs right over the axis of the tunnel, and decreases rapidly on either side of the tunnel. The magnitude of the settlements depends on the type of ground, the method of tunneling and the depth of the tunnel. The deeper the tunnel is, the smaller the settlements. However, the zone of influence of the tunnel settlements as represented by the settlement trough in Plate 9-2 increases as the depth of the tunnel increases. Shallow tunnels cause arger settlements than deep tunnels, but the width of the settlement trough is smaller for shallow

tunnels than for deep tunnels. As a general concept, the deeper the tunnels are, the smaller the settlements at the surface will be. When two tunnels are constructed close to each other, the settlements caused by each tunnel tend to be additive, as shown on Plate 9-2.

The settlements can be controlled by a variety of methods that involve treatment of the soils/rocks above the tunnel, and sometimes the soils through which the tunnel would pass to solidify them and create a canopy over the tunnel. Grouting is commonly used. Various grouting methods are available, including compaction grouting, chemical grouting, injection with microfine cement and jet grouting. Each method has its advantages and limitations.

The tunnels will be constructed through highly variable, sheared and weathered rocks, below the groundwater table. In addition, along a considerable part of the tunnel alignments the rock cover over the tunnels is small, typically less than 20 feet. In view of the highly sheared and fractured nature of the rocks, the stability of the face and the roof are of concern. Settlements would develop due to movements of the rocks and overlying soils. However, if dewatering (before tunneling begins) is not effective in lowering the groundwater table below the tunnel invert, settlements could also develop due to seepage into the tunnel and loss of ground as the fine-grained materials are washed with the groundwater flowing into the tunnels. The magnitude of the settlements is difficult to estimate because the natural processes that lead to the settlements cannot be modeled accurately. Assuming that good workmanship is employed in the tunnels and that strict procedures are followed to avoid problems caused by raveling of the roof, the settlements should be small, and should be limited within a zone about one tunnel diameter on either side of the axis of the tunnel.

The most serious concern associated with tunneling is the potential for excessive loss of ground at the roof or face of the tunnel that could lead to a cave-in. Such an occurrence could have serious impacts on buildings above the tunnel, and could pose a serious risk to the safety of the workers in the tunnels.

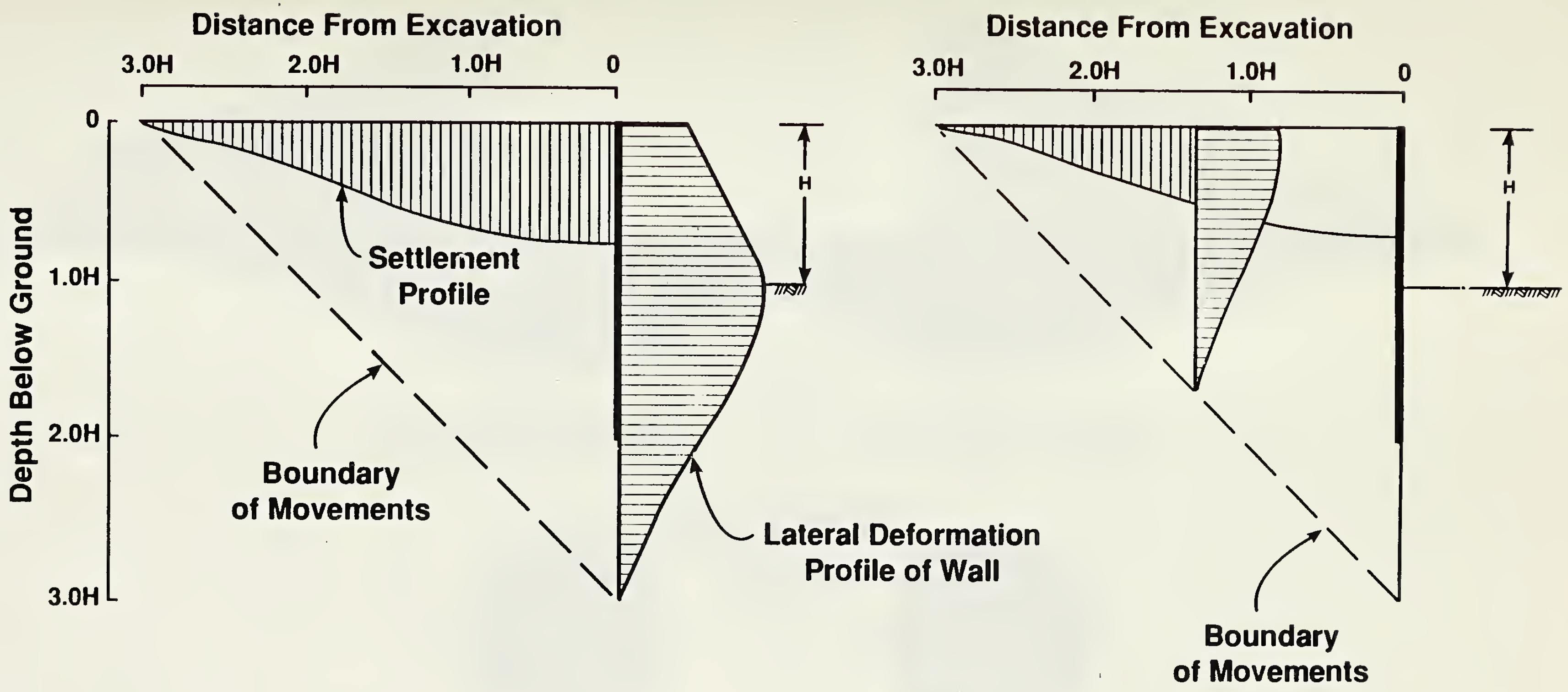
To minimize the potential adverse impacts of tunneling we recommend the following:

- The groundwater table be lowered before tunnel excavation begins, and maintained below tunnel invert until the permanent lining has been constructed.

- Tunneling be carried out by experienced miners and that strict excavation and shoring procedures be followed to avoid accidental cave-ins and collapse of the tunnels.

- A thorough monitoring program be implemented to verify that the groundwater table is indeed lowered to the required levels. Also, ground movements should be monitored to provide a basis for evaluating ground settlements and to provide a basis for anticipating conditions that could lead to excessive settlements or potentially to collapse of the tunnels, so that appropriate preventive measures can be implemented.

- During the detailed design phase of the project, a comprehensive exploration program should be carried out to map the areas where soft ground and potentially unstable conditions may be encountered. These areas should be treated where feasible to strengthen the in-situ soil and rock in advance of tunneling.



SETTLEMENTS AND LATERAL DEFORMATIONS
CAUSED BY EXCAVATIONS—
DEEP MUD PROFILE

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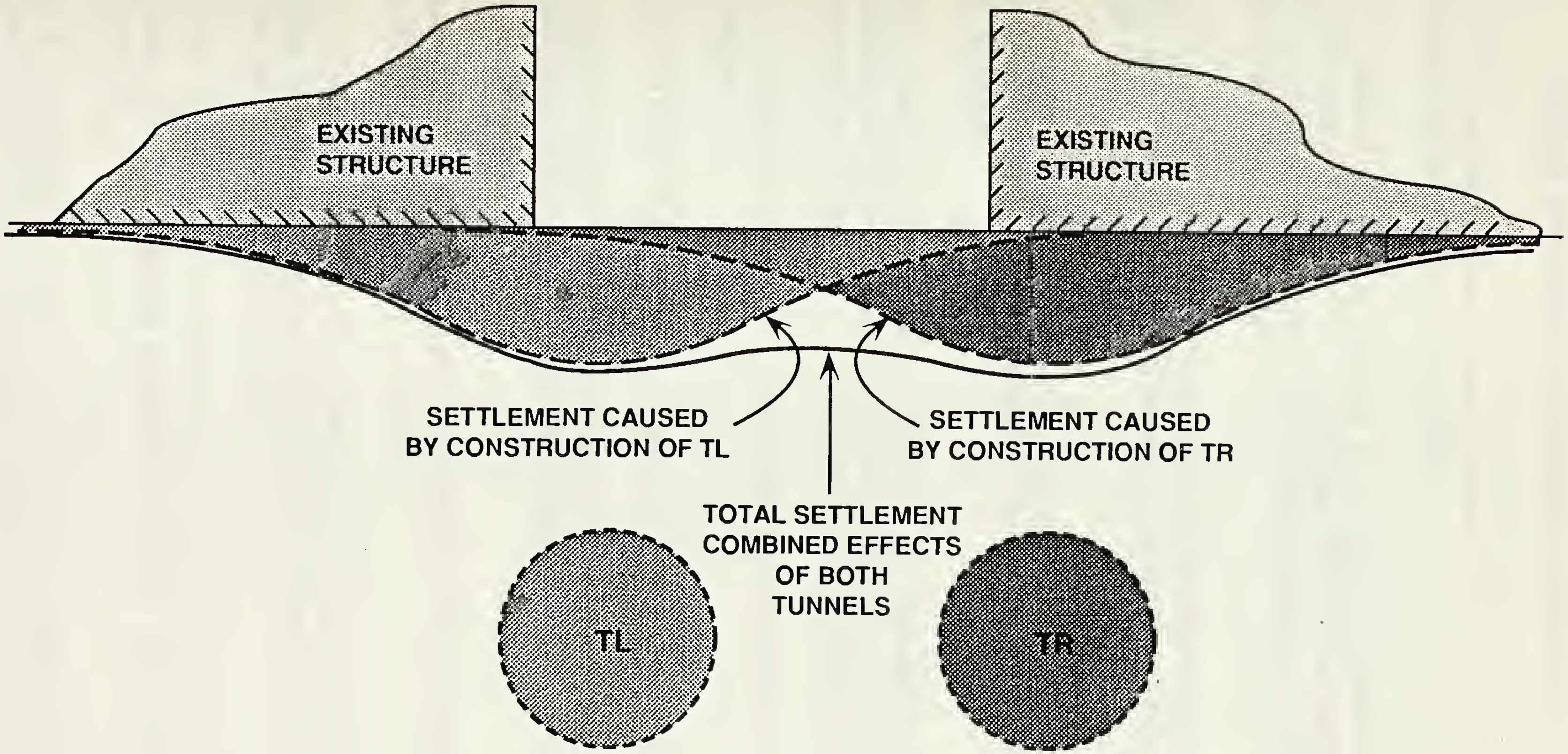
PLATE 9-1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SETTLEMENTS CAUSED BY TUNNELING

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PLATE 9-2

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APPENDIX

BOARD OF CONSULTANTS REPORTS

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APPENDIX

BOARD OF CONSULTANTS REPORT NO. 2

MEETING ON SEPTEMBER 14, 1995

APPENDIX

D. O'ROURKE **Geotechnical Consultant**

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-272-4029

XPRESS MAIL

27 September 1995

avid Minister
F Kaiser Engineers, Inc.
300 Harrison Street
akland, CA 94612

E: Board of Consultants Report Following the Meeting of September 14, 1995

ear David,

Enclosed please find a final version of the referenced report. By copy of this letter, the port is being sent to Demetrious Koutsoftas, Tor Brekke, and Norm Nadel.

The Board is pleased to be involved in the engineering and planning of this project.

Sincerely,

Tom O'Rourke

T.D. O'Rourke

nc.

: T. Brekke
D. Koutsoftas
N. Nadel

APPENDIX

September 27, 1995

CALTRAIN COMMUTER RAIL DOWNTOWN SAN FRANCISCO EXTENSION

REPORT FOLLOWING THE BOARD OF CONSULTANTS MEETING SEPTEMBER 14, 1995

Prepared by the Board of Consultants

This report has been prepared to summarize the major issues discussed and principal commendations made by the Board of Consultants (BOC) during its meeting of September 14, 95 at the offices of De Leuw Cather in San Francisco, CA. A list of those attending the eeting is available in Appendix A. A list of the BOC members is provided in Appendix B.

The Board was asked to comment on the technical feasibility, appropriate construction ethods, potential difficulties, and relative costs associated with underground construction for lternatives 2 and 3 for the CalTrain Commuter Rail Downtown San Francisco Extension. The oard's recommendations pertaining to Alternatives 2 and 3, as well as rock tunneling and soft ound tunneling, are summarized under the headings that follow.

LTERNATIVE 2

Alternative 2 covers six potential route configurations, all of which include a terminal ation on Beale St. The terminal station will involve one of three options: 1) stacked twin track atforms in an 83-ft-deep excavation between Market and Mission Sts. (2A-C), 2) staggered twin ack platforms in a 55-ft-deep cut on Beale St. and the Caltrains right-of-way (R/W) east of eale between Mission and Folsom Sts. (2B-C), and 3) four track center platform station in a 55-deep cut at the same location as the preceding option (2C-C). These three options, designated 2A-C, 2B-C, and 2C-C, involve cut-and-cover construction along Beale to the Embarcadero, me form of soft ground or cut-and-cover tunneling along the Embarcadero to Townsend St., d cut-and-cover construction along Townsend to the area near Fourth St. Three additional tions under Alternative 2 involve rock tunneling (2A-M, 2B-M, and 2C-M) from a location ar the intersection of Beale and Folsom Sts. on a broad curve to the intersection of Third and wnsend Sts.

The cut-and-cover construction for the various options of Alternative 2 is technically asible, and the support systems proposed by Dames & Moore for various sections of the routes e appropriate for stability and ground movement control.

APPENDIX

With the exception of the deepest cuts on Beale St. for the 3-level stacked station between pproximately Market and Mission, it has been proposed that the originally specified soldier pile emie concrete (SPTC) walls be replaced by soil mixed walls (SMWs) with heavy steel sections W 30 x 220) on 4-ft centers. The Board concurs with this recommendation, which will reduce e risk of slurry trench instability near the tip elevations of adjacent pile foundations, and notes at the use of SMWs will involve less expense than the SPTC walls. The Board also notes that, r cost estimating purposes, the SPTC walls have been assumed to provide only temporary xcavation support. In contrast to SMWs, which are suitable only for temporary support, SPTC alls can ben used in the permanent structural support system. At a future stage in the design, it ay be advantageous to evaluate SPTC walls as providing both temporary and permanent support r the underground railway structures.

The 55-ft-deep excavation on Beale St. and the Caltrains R/W between Mission and olsom Sts. will be 150 ft wide, with its east wall adjacent to the pile foundations of the Pacific ateway Building. Because of pile group bearing pressure at about the level of the Old Bay Mud, mbined with the depth of the adjacent excavation, the potential exists for significant settlement f the pile group. The Board, therefore, concurs with the proposed two stage excavation adjacent the Pacific Gateway Building. This approach provides a prudent basis for estimating the cost f construction in which the excavation procedure is designed to protect the adjacent building. he Board notes that further site characterization may disclose clay strength characteristics and le group bearing pressures more favorable than currently assumed such that a staged excavation proach may not be needed. Accordingly, the Board recommends that an upper and lower und estimate of cost be made for the construction in this area pertaining to a double and single age excavation, respectively. In this way, the costs can be bounded within a realistic range of tential contingencies.

The cut-and-cover construction between Colin P. Kelly, Jr. and Delancey Sts. and tween Third and Fourth Sts. is shown in the draft geotechnical report with SMW toe netration either in rock or 30 ft below the base of excavation. If dewatering outside the cavation is assumed as suggested in the report, a deep toe embedment would not be required r seepage control. It is likely under these conditions that a toe embedment of 15 to 20 ft will be itable for vertical wall stability and ground movement control.

The 83-ft-deep excavation on Beale St. associated with option 2A-C is currently proposed be 75 ft wide, and there is little separation between the SPTC walls and adjacent buildings. It ould be advantageous to provide at least a 3 ft separation between the exterior wall and building e to reduce the potential for building damage during SPTC construction. This adjustment will quire a terminal station excavation width of about 70 ft.

Option 2A-C will require a 75-ft wide excavation near the Harrison St. Bridge and the est anchorage of the San Francisco-Oakland Bay Bridge. The proximity of excavation to the

APPENDIX

foundations of the Harrison St. Bridge and the Bay Bridge anchorage will require special excavation and support procedures, as proposed by Dames & Moore. The Board concurs with these procedures, which near the Bay Bridge anchorage involve staged excavation and incremental support in approximately 25-ft-long sections adjacent to the anchorage. The Alternate B staged excavation procedure, explained at the BOC meeting, should be used for cost estimating purposes.

During the BOC meeting, it was disclosed that the basements of some high-rise buildings on Beale St. encroach on the R/W of the 83-ft-deep excavation. Such encroachment will involve additional difficulties and costs associated with access, alteration, and potential underpinning and emoval of basements. The Board believes that option 2A-C is the least attractive of the cut-and-cover operations because of these encroachments, and because of the considerable depth of station excavation and wide excavations near the Harrison St. Bridge and Bay Bridge anchorage. Option 2B-C would encounter similar difficulties even though it is shallower.

In contrast, Option 2C-C allows for excavation of considerably less depth. The Board ecommends that the width of excavation be reduced to 42 ft in the vicinity of the Harrison St. Bridge and Bay Bridge anchorage. In addition, the Board recommends both that the excavation base be raised as much as 15 ft and that the excavation be shifted as far east as possible away from he Bay Bridge anchorage. These adjustments in depth and plan should place the anchorage outside the zone of influence for significant excavation effects on the structural and functional ntegrity of the anchorage. With appropriate changes in width, depth, and plan location near the Bay Bridge anchorage and Harrison St. Bridge, Option 2C-C is the most advantageous of the cut-and-cover options.

Consistent with the Board's recommendations following the July 6, 1995 BOC meeting, he rock tunneling options 2A-M, 2B-M, and 2C-M are all subject to very difficult tunneling onditions along the broad curve from the intersections of Folsom and Beale Sts. to Third and Townsend Sts. These tunneling conditions are associated with significant risk of undermining overlying structures and of incurring substantial expense, as explained under the section of this eport entitled "Rock Tunneling".

Several soft ground tunneling approaches have been considered along the Embarcadero oadway. Various tunneling options for this portion of Alternative 2 are discussed in the section f this report entitled "Soft Ground Tunneling".

ALTERNATIVE 3

Alternative 3 covers four potential route configurations from Townsend St., all of which nclude a terminal station on the site of the existing Transbay Transit Terminal. Two of the four oute and terminal options involve above and below ground train access at the Transbay Terminal 3A-C and 3B-C, respectively), cut-and-cover construction along most of the route, and an

APPENDIX

pproximate 1400-ft-long section of rock tunnel from a point near the intersection of Folsom and ssex Sts. to a location between Brannan and Bryant Sts. near Colin P. Kelly, Jr. St. The two dditional options under Alternative 3 (3A-M, and 3B-M, respectively) involve rock tunneling om the intersection of Folsom and Essex Sts. to the intersection of Third and Townsend Sts., in ombination with above and below ground train access at the Transbay Terminal.

The cut-and-cover construction for the various options of Alternative 3 is technically easible and the support systems proposed by Dames & Moore for various sections of the routes re appropriate for stability and ground movement control. As a clarification, the Board notes hat the excavation width along Colin P. Kelly, Jr. St. is planned as 42 ft. This width should be hecked to confirm that it provides sufficient separation between the excavation wall and uildings for work to proceed without damaging the building facades.

Consistent with the BOC recommendations following the July 6, 1995 meeting, the 2500- -long rock tunneling along the curved portion of the route which connects the intersections of olsom and Essex Sts. to Third and Townsend Sts., associated with options 3A-M and 3B-M, arries significant risk of undermining overlying structures and incurring substantial expense. In ontrast, limiting the length of rock tunneling to only 1400 ft from the intersection of Folsom and ssex Sts. to a location between Brannan and Bryant Sts. (Options 3A-C and 3B-C) confines the ifficult conditions to a relatively short section of the route where tunneling will be performed eneath only two or three buildings plus the I-80 freeway. Selecting Option 3B-C would increase he tunnel invert depth, thereby increasing the rock cover above the tunnel crown under Rincon ill to larger and more desirable levels. Further discussion of rock tunneling is provided in the ext section.

The 3A-C and 3B-C railway routes would follow a curve with an approximate 400-ft adius from Townsend to Colin P. Kelly, Jr. St., where it would pass beneath two 100-year old, istoric brick masonry buildings. It is not advisable to drive a tunnel under mixed face conditions eneath these buildings. A more advantageous approach would involve underpinning the ructures in conjunction with cut-and-cover construction. Reinforcing the building foundations nd underpinning with needle beams and/or special frames are technically feasible and less costly nd risky than mixed face tunneling.

OCK TUNNELING

Although “rock tunneling” is discussed in this report, the Board uses this term cautiously. he term should not create a false impression regarding the nature of the tunneling operation, hich will involve construction in both “rock-like” and “soil-like” materials.

The geology along the tunnels varies considerably, and includes sands, clays, weathered nd fractured sandstones (graywackes) and shales, and shear zones. Thus a mixed face condition

APPENDIX

at hand because geological materials of quite different properties will be present in the tunnel ce at one time. Further, the materials are by-and-large of such poor quality for tunneling that artial face excavation (i.e., only part of the tunnel cross-section) must be performed to minimize e excavated face area to be able to prevent the loss of ground. Such loss could readily ropagate to the surface, causing severe impacts on structures, streets, and utilities.

Under these conditions, the Board believes that mining a single double track tunnel would e preferable to excavating two single track tunnels. The difficulty of maintaining the relatively arrow unexcavated pillar between twin tunnels, and the efficiency that results from the reduction f the number of excavation and support passes in the case of the single tunnel are important ctors in favor of the single double track tunnel. It is recommended, therefore, that the ngineering plans focus on a single, double track tunnel rather than twin tunnels.

During the July 6, 1995 meeting, a tunneling procedure for a single double track tunnel as discussed in which a center drift and two wall plate drifts could be excavated in the tunnel eading, followed by sequential excavation and support of the left and right sides of the heading. he initial drifts at the top of the tunnel can be used for exploration purposes, as well as for cess points for grouting and dewatering. The remainder of the tunnel face could be excavated y removing the bench while providing support for the sidewalls. This sequence of work was tended to illustrate the phased excavation and incremental support process, and as an aid to timating potential costs. Alternative excavating procedures may be equally valid.

Particular attention is required to the means by which the lower side walls can be pported in earth or in very poor rock. Rock bolts can be an effective tool to support tunnel penings. The mixed and fractured nature of the ground, however, may make them unreliable for e proposed work. To the extent rock bolts can be employed, particularly for the temporary pport of tunnel sidewalls, Swellex bolts are recommended. They can be installed quickly and liably.

It is probable that portions of the partial face excavation can be carried out using a echanical excavator, such as a roadheader. It would be ideal if all of the rock excavation could performed in this manner. However, the strength of some of the sandstones, and the limited acturing at some locations may limit excavation in this fashion. Blasting may be the only fective way to perform some of the excavation when zones of competent sandstone are countered.

Because of the presence of sands and soft clays in the face at some locations, control of oundwater is essential to keep these materials from flowing into the tunnel excavation. Wells n be installed to a depth below the tunnel invert and pumping started two to three months in vance of the tunnel excavation. However, where the sands overlay relatively impervious aterials, such as clay or rock, it is impossible to dewater totally the sands. The best that could

APPENDIX

e accomplished in these circumstances is the reduction of the water pressure in the sands. ining in the vicinity of these sands will be very slow, and will require great care on the part of e miners to prevent loss of ground. It is difficult these days to assemble a crew of people with e requisite skills and experience.

Installation of dewatering wells along the route may prove to be infeasible unless owners f private properties grant access for installation and operation of the wells. Dewatering from the nnels may be feasible, but extraordinarily costly because of the interference it causes to the nnel excavation. Such dewatering is also limited by the inability to dewater the sands over npervious materials. Dewatering, whether conducted from within the tunnel or from above, may epress ground water tables locally and cause consolidation of soils such as the Recent Bay Mud. ettlements of structures, street surfaces, and utilities can result unless special measures, such as nderpinning or compaction grouting, are employed.

As an alternative to dewatering, ground freezing, grouting, and compressed air can be nployed. Ground freezing and grouting are very expensive and slow, particularly if they must be rformed from within the tunnel. Provision for the use of compressed air should be made at the tset, even if its use may not be ultimately required. In this way, if needed, it can be mobilized ickly, and the tunnel work can be advanced expeditiously. The problems associated with the pulsion of gasses and vapors, which is discussed with respect to the soft ground tunnels at the mbarcadero, are less likely to be as severe in the vicinity of the rock tunnels being considered. onetheless, variations of ground water elevation and permeability of soils along the alignment uld lead to air losses that may force toxic gasses or fumes from contaminated soils and rock to adjacent buildings.

It follows from the foregoing discussion that it is advantageous to have as much rock ver as possible above the crown of the tunnel. Except for the tunnel from the intersection of lsom and Essex Sts. to a location between Bryant and Brannan Sts. (Options 3A-C and 3B-C), sufficient rock cover will be a problem for significant portions of the proposed rock tunnel ctions. Tunneling for Options 3A-C and 3B-C will still be difficult, but the rock tunneling in ese cases will be confined to relatively short length, which has substantial rock cover and is cated below only three buildings. As explained previously, Option 3B-C would increase the ck cover relative to Option 3A-C, thereby providing for more advantageous tunneling nditions.

Estimating tunneling costs is a highly subjective enterprise. It is far from an exact science. equently, when experienced contractors submit bids for tunnel work which has been designed in eat detail, the variation in the bids is substantial. The differences are the results of the bidders' ferent perceptions of the difficulties which will be encountered. Accordingly, at this stage of nning, it is very difficult to provide reliable estimates of tunnel costs. In contrast, cost timates for cut-and-cover will be significantly more reliable and less sensitive to uncertainties sociated with ground conditions and effects on adjacent property.

APPENDIX

During the September 14, 1995 BOC meeting, the construction of a 75-ft-wide, four-track unnel for the 2A-M option was discussed. This section of rock tunneling would be pproximately 600 ft long from the intersection of Harrison and Beale Sts. southwest to the I-80 eeway. The staged excavation and incremental support procedures proposed for this section of unnel represent an appropriate means of estimating the construction costs. Because of the poor uality of the rock, this section of the tunnel will be very difficult to excavate and stabilize. The roblems associated with rock support and ground movement control, in combination with the ubstantial expense of the staged excavation and incremental support procedures, are significant isadvantages for this option.

OFT GROUND TUNNELING

Twin soft ground tunnels are being considered for about 1000 ft of the route under the mbarcadero as part of options 2A-C, 2B-C, and 2C-C. The surface area where the tunnels are be driven was created by filling in the bay. During its history, there were wharves and docks in e area where ships may have been sunk and abandoned in place. Certain to be encountered will e timber piles which were used to support the waterfront structures. The nature of the soil and s location below the ground water table make it essential to stabilize the face of the tunnels while ey are being excavated.

During the September 14, 1995 BOC meeting, the possibility of using an earth balance ield was discussed. If an earth balance shield were to encounter buried piles and ships, ubstantial problems would result with respect to stabilizing the tunnel face and disrupting the unneling operation. Although the Board believes it is unlikely that an earth balance shield will be fective here, it nevertheless would be helpful to obtain historic information about previous ructures in the area. The historic information may show locations with a high likelihood of piles nd buried debris from which a more detailed assessment of earth balance shield feasibility could e made.

Another option may involve the strengthening of the ground by jet grouting from the urface. Apart from its apparently great expense, this has the disadvantage of causing onsiderable disruption to activities at street level along the Embarcadero. It is advisable to btain additional information about jet grouting operations and their environmental consequences om a knowledgeable contractor who has substantial experience with the method. Furthermore, will be advantageous to obtain information on the costs of jet grouting directly from the San ancisco Department of Public Works in relation to its recent application during construction of e Islais Creek Transport/Storage Facilities.

The use of compressed air in the tunnels is another possibility, but its use can cause the pulsion from the ground into the surrounding buildings and nearby vicinity of unpleasant and ossibly noxious gasses and vapors. Many such problems occurred during recent compressed air

APPENDIX

tunneling in San Francisco. Compressed air is also very expensive and can involve some physical risk to the workers and to people in the neighboring structures.

As discussed at the September 14, 1995 BOC meeting, the most advantageous option in his area may be what was called "environmentally sensitive" cut-and-cover (ESCC) construction. he ESCC approach would involve the use of concrete beams or panels as covers that are placed lush with current street and sidewalk grades. These features have been incorporated on a short ection of the Muni Metro Turnaround, and also on recent sections of the L.A. Metro. Open cuts r shafts for transportation of muck, tunnelers, and materials would be located away from this ensitive area. Moreover, certain construction operations could be limited to specific times of the ay, and noise controls implemented through the use of special equipment. If the ESCC approach s adopted, the Alternative 2 route would end near the intersection of Beale and Mission Sts. with ut-and-cover construction along the Embarcadero.

ONCLUDING REMARKS

On the basis of the findings and considerations discussed above, it is the opinion of the oard that certain options are most advantageous with respect to other options regarding the pplication of underground technology and protection of adjacent structures. For Alternative 2, ption 2C-C is most advantageous, especially if the excavation is shifted as far east as possible way from the Bay Bridge anchorage and its base is raised by approximately 15 ft in this vicinity. or Alternative 3, Option 3B-C is the most advantageous for route construction to a new ransbay Transit Terminal.

APPENDIX

APPENDIX A

APPENDIX

Appendix A
Attendance at Board of Consultants Meeting

CALTRAIN SPECIAL
BOARD OF CONSULTANTS MEETING

14 September 1995

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APPENDIX

APPENDIX B

APPENDIX

Appendix B
Board of Consultants

SPECIAL BOARD OF CONSULTANTS

CALTRAIN COMMUTER RAIL DOWNTOWN SAN FRANCISCO EXTENSION

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APPENDIX

BOARD OF CONSULTANTS REPORT NO. 1

MEETING ON JULY 6, 1995

APPENDIX

. O'ROURKE **Geotechnical Consultant**

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PRESS MAIL

27 July 1995

d Minister
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E Board of Consultants Report Following the Meeting of July 6, 1995

David,

Enclosed please find the final version of the referenced report. With copy of this letter, eport is being sent to Demetrious Koutsoftas, Tor Brekke, and Norm Nadel.

We look forward to seeing you at the September 14, 1995 Board of Consultants' Meeting n Francisco.

Sincerely,

Tom O'Rourke

T.D. O'Rourke

lc

T. Brekke
D. Koutsoftas
N. Nadel

APPENDIX

July 27, 1995

CALTRAIN COMMUTER RAIL DOWNTOWN
SAN FRANCISCO EXTENSION

REPORT FOLLOWING THE
BOARD OF CONSULTANTS MEETING
JULY 6, 1995

Prepared by the Board of Consultants

This report has been prepared to summarize the major issues discussed and principal mmendations made by the Board of Consultants during its meeting of July 6, 1995 at the es of De Leuw Cather in San Francisco, CA. A list of those attending the meeting is available ppendix A. A list of the Board Members is provided in Appendix B.

The Board was asked to comment on the technical feasibility, appropriate construction ods, potential difficulties, and relative costs associated with underground construction for rnatives 2A, 2C, and 3A for the CalTrain Commuter Rail Downtown San Francisco nsion. The Board's recommendations pertaining to Alternatives 2A, 2C, and 3A, as well as eling procedures and costs, are summarized under the headings that follow.

ERNATIVE 2A

The cut-and-cover construction along the route of Alternative 2A is technically feasible, the support systems proposed by Dames & Moore for various sections of the route are opriate for stability and ground deformation control. The deepest excavation, which is along e St. between Market and Howard Sts., can be constructed as proposed with solider pile ie concrete (SPTC) walls and cross-lot braces. This type of support system has been used cessfully in similar conditions on other projects, including the BART Embarcadero Station, e support was provided for a 70-ft-deep excavation in very similar soils. Although the mum depth of the proposed CalTrain excavation is 83 ft versus 70 ft for the Embarcadero on, the additional depth will be achieved by extending the excavation into Old Bay Clay. material has sufficient strength and stiffness in combination with the SPTC support system to it the additional excavation without compromising base stability or increasing wall mation beyond permissible levels. In some locations, such as 50 Beale St., the proposed vation will be taken below the bottom of pile foundations supporting high-rise buildings. In locations, stabilization and stiffening of basal soils by jet grouting has been proposed to ide additional security for the adjacent foundations. This procedure appears to be feasible for roposed excavation, and has been used successfully on other projects in San Francisco, bly Contract E for the Islais Creek Transport/Storage Facilities.

APPENDIX

Between Folsom and Harrison Sts. along Beale St., eight levels of cross-lot braces have bn proposed. Because of the absence of Recent Bay Mud at this location, the scarcity of high-ri buildings, and reduced density of adjacent structures, it appears that suitable control of gund deformation can be achieved with 6 or 7 brace levels.

Although careful excavation and support will be required in the vicinity of the Bay Bridge ahorage, cut-and-cover construction adjacent to the anchorage is technically feasible without cnpromising the stability or serviceability of the bridge. It would be advantageous, nonetheless, tposition the excavation as far from the anchorage as practical.

Some of the most costly and difficult cut-and-cover construction is proposed from Market tFolsom Sts. along Beale St. where the deepest excavation will be located in close proximity to hh-rise buildings. Moreover, the excavation in this area will be made through a relatively thick laer of Recent Bay Mud. Because the Recent Bay Mud is soft and deformable, special care will bneeded for ground movement control.

It would be very advantageous to reduce the depth of excavation in this area through use oexisting land near the TransBay Terminal. If the width of the train corridor could be increased tough construction on property adjacent to Beale St. between Folsom and Mission Sts., the dth of excavation in this area could be decreased from 83 ft to approximately 55 ft. This aproach would involve terminating the CalTrain station at Mission St., thereby eliminating the dp cut-and-cover construction from Mission to Market Sts. An underground people mover a/or pedestrian walkway could be built to link the CalTrain station with BART. Removing the depest and most difficult city block of cut-and-cover tunneling from the construction scheme and eucing the depth of excavation would have substantial ramifications on cost. It is likely that sh an approach could save from $50 million to $100 million in construction expense relative to th associated with the current plan for Alternative 2A.

When performing the cut-and-cover construction, it will be advantageous to provide for ecation of utilities under a separate contract. This will allow for specialty contractors to peorm this phase of the work, and thereby contribute to improved efficiency in the overall cstruction process.

ATERNATIVE 2C

This option currently includes the construction of a single, double track or two single rk tunnels on a broad curve from the intersection of Harrison and Beale Sts. to the intersection of ownsend and 3rd Sts. The geology along the tunnel varies significantly, and includes sands, ts, weathered and fractured sandstones (graywackes) and shales, and shear zones. Thus, a mied face condition is at hand because geological materials of quite different properties will be rent in the tunnel face at any one time. Further, the materials are by-and-large of such poor

APPENDIX

lity for tunneling that partial face excavation (i.e., only part of the tunnel cross-section) must performed to minimize the chance for loss of ground. Such loss could readily propagate to the ace, causing severe impacts on structures, streets, and utilities.

Because this type of tunneling will need to secure each increment of excavation before ceeding to the next, it will be more efficient to drive a single, double track tunnel rather than eat the phased excavation and incremental support process in two separate drives. It is ommended, therefore, that the engineering plans focus on a single, double track tunnel.

The partial face excavation can be carried out using a mechanical excavator, such as a dheader, immediately followed by incremental support to stabilize the ground. It would be l if all the rock excavation could be performed by a roadheader. However, the strength of e of the sandstones and the limited fracturing at some locations may prevent excavation in this sion. Blasting may be the only effective way to perform some of the excavation.

To make the procedures outlined above feasible, control of the groundwater is essential. mally, this is accomplished by installing wells to a depth below tunnel invert some 40 to 90 a before the tunnel approaches. Here, this may prove to be infeasible unless owners of private perties along the alignment grant access for installation and operation of the wells. atering from the tunnel itself is feasible, but extraordinarily costly because it severely slows n tunnel advance. The same is true for grouting and freezing of the ground ahead of the nel face. Dewatering, either external to or within the tunnel, may depress groundwater levels cly and cause consolidation of soils, such as the Recent Bay Mud.

Provision for the use of compressed air should be made at the outset, even if its use may ultimately be required. In this way, if it should be required, it can be mobilized quickly, hout causing an untoward delay in the performance of the work. Moreover, variations of the ndwater elevation and permeability of soils along the alignment could lead to air losses that ld force toxic gases or fumes from contaminated soils or rock into adjacent buildings.

It must be noted that groundwater control is not necessary in terms of keeping the tunnel y However, it is essential to prevent the water from bringing materials, such as sand or silt, the tunnel, causing instability and loss of ground control.

Should Alternative 2C be pursued, it must be understood that it will require miners and gineers that have outstanding skills and experience. It is very difficult to find such teams these ay

Alternative 2C carries high potential risk with respect to instability of the overlying soil d rock, and the impact that a sudden loss of ground could have on the foundations of overlying ildings. Moreover, there are substantial uncertainties at this stage of planning with respect to ffects of variable ground conditions on the cost of a mined tunnel.

APPENDIX

TERNATIVE 3A

The cut-and-cover construction proposed for this route is feasible, and the proposed xavation support systems are suitable for stability and ground deformation control. A mined nel is proposed along the northward projection of Colin P. Kelly St. from Bryant to Folsom t In the area of the proposed tunnel between Bryant to Brannan Sts., sufficient rock cover pears to be lacking. Accordingly, it is advisable to lower the invert elevation of the tunnel at location by as much as the geometric configuration of the alignment will allow, but not less n 20 ft, to provide rock cover between the tunnel crown and foundations of the overlying dings. Reduction of the invert elevation will increase the depth of cut-and-cover excavation ng Colin P. Kelly and Townsend Sts., with a corresponding increase in construction cost in e areas. Mined tunneling along this route is subject to constraints, difficulties, and risks lar to those described for Alternative 2C. Depending on the final elevations of invert chosen Alternative 3A, a significant part of the mined tunneling may be above the regional undwater table (at approximately El. 0). This situation could reduce the risk of water in-flow n sudden loss of ground during tunneling, although it will not eliminate the possibility of xpected infiltration from locally perched water.

NNELING PROCEDURES AND COSTS

Estimating tunneling costs is a highly subjective enterprise. It is far from an exact science. quently, when experienced contractors submit bids for tunnel work, the variation in the bids substantial. The differences are the results of the bidders' different perceptions of the culties which will be experienced. Accordingly, at this stage of planning, it is very difficult to vide reliable estimates of the tunneling expenses. In contrast, cost estimates for cut-and-cover xavation will be significantly more reliable and less sensitive to uncertainties associated with und conditions and effects on adjacent property.

During the July 6, 1995 meeting, a tunneling procedure for a single, double track tunnel discussed in which a center drift and two wall plate drifts could be excavated and supported e tunnel heading, followed by sequential excavation and support of the left and right sides of heading. The remainder of the tunnel face could be excavated by removing the bench in two ges while providing support for the sidewalls. This sequence of work was intended to illustrate phased excavation and incremental support process, and as an aid to estimating potential s. This tunneling procedure is only a preliminary approach, and alternative procedures may be ally valid and perhaps more advantageous. Alternative tunneling schemes and more surface data will require further evaluation before an acceptable tunneling method can be cted with confidence for this work. Particular attention is required for the means by which the er side walls can be supported in earth or in very poor rock. The initial drifts at the top of the nel can be used for exploration purposes as well as for access points for grouting and watering.

APPENDIX

To maximize the cover over the top of the tunnel, it may be advisable to employ circular s which are not installed in a full semi-circle to support the roof. Flattening the arch in this sion has the disadvantage of increasing the stress in the ribs. Assuming the span is 40 ft, the of the rib at the center is 10 ft, and the superimposed ground load is 4.0 kips/ft^2, the axial to be carried by the ribs is approximately 80 kips/ft of tunnel. If ribs are placed on 4 ft rs, the axial load per rib is 320 kips. Further assuming that continuous shotcrete is used to rfer load, ribs weighing approximately 80 pounds per lineal foot of rib are required. [See or and White, (1946, 1977) for more detail.]

Rock bolts can be an effective tool to support tunnel openings. The mixed and fractured e of the ground, however, may make them unreliable for the proposed work. To the extent bolts can be employed, particularly for the temporary support of tunnel sidewalls, Swellex are recommended. They can be installed quickly and reliably.

At the meeting, there was some discussion of tunneling advance rates and crew sizes. It d be emphasized that the advance rates discussed were applicable to work performed after arning curve for the given operation. Average rates will be significantly less. Crew sizes applicable only to the specific operation in question. In addition, there are people required very job who service the job generally. Examples might be mechanics, electricians, ressor operators, crane operators, truck drivers, and so forth. In addition, professional eers are required for a variety of rolls, as well as construction superintendents, clerks, and keepers.

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r, R.V. and T.L. White, “Rock Tunneling with Steel Supports”, Commercial Shearing and mp Co., Youngstown, OH, 1946 (revised 1968).

r, R.V. and T.L. White, “Soil Tunneling with Steel Supports”, Commercial Shearing, Inc., Youngstown, OH, 1977.

APPENDIX

APPENDIX A

Attendance at Board of
Consultants Meeting

APPENDIX

BOARD OF CONSULTANTS MEETING

6 JULY 1995

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APPENDIX